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A JOURNAL OF HIGHWAY RESEARCH

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Numerous Grade Crossings Retard the Flow of Traffic on the Cook County Highways

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U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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THE COOK COUNTY TRANSPORTATION SURVEY

A Digest of the Report of a Study of Highway Traffic and the Highway System of Cook County, Illinois[1]

Reported by J. Gordon McKay, Chief of the Division of Highway Transport and Economics, U. S. Bureau of Public Roads

ANALYSIS of the present volume of traffic on the highways of Cook County, the composition of this traffic, and the expected traffic in 1930 demonstrates the need for extensive highway improvements in the county during the next few years. Highway traffic in sections of the county, particularly the zone surrounding the city of Chicago, already exceeds the functional capacity of the improved routes in these sections, and the great increase in traffic that may be expected during the next five years indicates that unless a comprehensive betterment program is initiated and substantially completed within this period, the county and the city of Chicago will suffer seriously from the lack of adequate highway facilities. The daily cost of highway congestion to the citizens of Cook County is excessive at the present time, and will become enormous by 1930, unless additional highway improvements are made in the county and the city.

THIS article is a digest of a report containing the results of highway traffic studies in Cook County, Ill., conducted during the summer and fall of 1924 under the cooperative research agreement between the Bureau of Public Roads, United States Department of Agriculture, and the Cook County highway department.

These investigations were undertaken in order to obtain essential facts concerning traffic on the Cook County highways as a basis for planning the development of the highway system in the Chicago regional area to serve present and future traffic.

The conclusions are based upon the present density, type, loading, and distribution of traffic units on the highways in the area, upon present population and population trends, upon predicted future traffic, and upon an economic and physical analysis of other factors affecting a highway improvement program.

The full report contains the data and conclusions and presents a recommended plan of highway improvement for Cook County indorsed by the Cook County highway department, the United States Bureau of Public Roads, the Illinois Department of Highways, and the Chicago Plan Commission. This program is an integral part of the plan of development for the regional area, which includes highways and streets under the jurisdiction of the several agencies.

It is essential to formulate a coordinated plan of highway improvement which will provide highway transportation service for present and expected future traffic. The present system of improved highways is unsatisfactory for present traffic and an analysis of the reasons for the inadequacy of the present system will be of value in developing the plan of highway improvement to fully serve present and expected future traffic.

PRESENT HIGHWAY SYSTEM INADEQUATE

The present highway system is, to a large extent, a development of a system of roads established during the period when the horse-drawn vehicle was the vehicle of highway transportation. A considerable mileage of these roads have been provided with high-type surfaces, but the locations have not been changed and the system is now inadequate for present traffic.

Population and industrial development in the county have increased markedly during the period of rapid growth in motor vehicle usage, particularly in the area adjacent to the city of Chicago. This growth, with the closely related increase in motor vehicles, has resulted in a steadily increasing volume of traffic on the highways of the county and city.

The building of highways, in spite of a continuous construction program, has not kept pace with the increasing volume of traffic. During the period 1913 to 1925 there were constructed in the county 346 miles of paved highways at a cost of approximately $10,440,000, nearly 1 mile of improved highway for every 2 square miles of area. Since 1920 the mileage of improved highway per registered motor vehicle has been steadily decreasing.

The present highway system is unable to serve the present traffic properly not only because of the lack of improved mileage but also because of the lack of adjustment of the present system to the traffic needs. The analysis of the 1924 traffic shows the importance of the city of Chicago as a source of the traffic on the highways of the county. A very large part of this traffic consists of vehicles operating between Chicago and other cities and villages in the county. As the cities and villages located within 5 miles of Chicago include 74 per cent of the total population of the county, exclusive of Chicago, it is evident that the greatest volume of traffic must be between the city and the 5-mile zone surrounding the city. There are, however, more improved highway routes leading to Chicago at some distance from the city line than there are at the city line because of the convergence of main routes as they approach the city.

LACK OF CONTINUOUS ROUTES IN CITY AND COUNTY

In the city there is a lack of continuous through routes upon which traffic can move freely and rapidly within the city and between the various sections of the city and county. The effective use of the county highway system is impaired by the lack of a sufficient number of connecting streets within the city; and this lack of coordination of county highways and city streets is an important cause of traffic congestion.

Traffic between the cities and villages of the county is second in importance to traffic between these cities and villages and Chicago. Highways connecting these cities and villages are few in number and unable to meet present traffic needs. A network of improved highways connecting them will provide routes for local traffic and relieve congestion at the city entrances by permitting through traffic to "by-pass" the city.

The traffic capacity of the present county system is also impaired by the lack of continuous, direct, improved routes. There is no continuous east-west highway across the county between the Lincoln Highway,

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[1] The highway traffic studies upon which the report is based were conducted under the joint supervision of Thomas H. MacDonald, Chief of the Bureau of Public Roads and Maj. George H. Quinlan, Superintendent of Highways of Cook County. J. Gordon McKay, Chief of the Division of Highway Transport and Economics, Bureau of Public Roads, directed the work of the survey and the preparaton of the report assisted by O. M. Elvehjem, E. T. Stein, and L. E. Peabody, all of the Bureau of Public Roads, and Warner Harwood of the Cook County highway department.

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which crosses the southern part of the county, and One hundred and eleventh Street, 12 miles to the north. By the development of 7½ miles of new highway in this section it will be possible to provide five additional continuous east-west routes. A few miles of new road to close the gaps in the north-south highways will open three additional through routes. The same condition exists in other sections of the county. By paving approximately 680 miles of new highways a large number of present non through routes will be connected and converted into through routes, and the traffic capacity of the highway system greatly increased.

NATURAL AND ARTIFICIAL BARRIERS IMPEDE HIGHWAY DEVELOPMENT

The natural and logical development of the highway system has also been impeded by the presence of certain barriers and obstructions to its orderly development. These barriers are of two types—natural and artificial.

Natural barriers are limited largely to waterways, the topography in other respects being favorable to highway location. Extending southwest from the city are three parallel waterways,—the Des Plaines River, the Sanitary Canal, and the old Illinois and Michigan Canal. Bridge construction over these waterways is very expensive and has had considerable influence on highway location. At the present time there are only three locations where highways cross these waterways in a distance of 20 miles.

Lake Calumet and Wolf Lake are also serious water barriers to highway planning. The lack of connecting highways between the city and the southeastern part of the county through the area occupied by these lakes and the surrounding lowlands has compelled traffic that would enter or leave the city via such routes to use either Indianapolis Avenue or the highways west of the lake district. This results in increased motor-vehicle operating costs due to the indirect routing and also congests the present entrances to the city.

Although natural barriers to highway development in the county are few, artificial obstructions are numerous. The latter are the natural result of the growth of Chicago and the surrounding area.

Railroad trunk lines enter the city from all directions. Industrial trackage, freight yards, classification yards, and structures essential to the railroad business surround the city on all sides. These, together with industrial establishments, form complete barriers to highway routes. Highways must be constructed around rather than through such areas. Crossings at grade reduce the traffic capacity of the routes and are a constant hazard to vehicular traffic.

A similar effect upon highway development is caused by the many special-use areas which have developed in the suburban area around the city. Brickyards, quarries, golf courses, cemeteries, and other developments requiring large acreages are obstacles to the construction of through highways.

The rapid growth of population and motor vehicle registration, the decrease of improved highway mileage in proportion to motor vehicle registration, the lack of improved city connections and sufficient through routes in the city, the lack of adjustment of the present highway system to traffic needs, gaps in the present highway system, and the natural and artificial barriers to highway development are the principal causes of the inadequacy of the present highway system and of traffic congestion.

The traffic congestion results in an enormous loss of time. Its elimination would result in a great saving both to vehicle owners and the public. It is difficult to estimate the cost, but if the loss due to delays in the transportation of people and commodities and the additional use of gasoline and oil due to traffic delays could be computed the total would be very large.

During an eight-hour period at a railroad crossing on Western Avenue, where traffic exceeds 15,000 vehicles per day, the crossing was blocked an average of 17 minutes per hour, and during one hour was closed 30 minutes.

The utilization in 1924 of 418 miles of highway in the county was approximately 990,000 vehicle-miles per day. Assuming an average speed of 20 miles per hour, the daily vehicle-hours are 49,500. Assuming that each vehicle loses four minutes per hour, due to traffic congestion on the county highways caused by natural or artificial obstructions or indirect routes, there is a daily time loss to traffic of 3,300 vehicle-hours. Considering the number of persons per passenger car and the net load per motor truck, the value of this lost time is at least $3 per hour. On this basis, there is a daily net loss, due to traffic delays, of $9,900 on these 418 miles of highway. Assuming this condition to exist 300 days during the year, the annual loss is approximately $3,000,000.

Recognizing the seriousness of the situation, the Cook County commissioners requested that the United States Bureau of Public Roads cooperate with the county in an investigation of highway traffic and in the formulation of a plan of improvement.

PURPOSE OF AND SCOPE OF THE TRANSPORTATION SURVEY

The principal purpose of the survey was to determine the present and anticipate the future traffic importance of all highways in the county as a basis for the designation of the highways to be included in a comprehensive plan of improvement.

The second major purpose was to classify the selected highways in accordance with their relative traffic importance and thereby to determine the relative necessity for and order of the several improvements; after which the third purpose was to determine the most economical type, design, and width for the improvement of the several sections on the basis of the following factors:

1. Present and estimated future traffic density.
2. The average daily number of motor trucks of large, medium, and small capacity using various sections of the system.
3. Maximum wheel loads and the frequency of heavy gross loads and wheel loads.

In general, these factors govern the choice of the type of pavement and the design and width of the improvement, though the final planning of each specific improvement must consider financial and physical factors such as the cost of the improvement in relation to traffic use, topography, drainage, soil and subgrade conditions, and availability and cost of materials.

For the purpose of securing this information 73 traffic stations were located throughout the city and county. At 17 of these, motor-truck weights were recorded. Recording stations were grouped around the weight stations and the truck weights recorded at each of the latter were imputed to the surrounding stations of the former class. At all stations information as to origin, destination type of vehicle, character of load, and purpose of use was recorded for both passenger vehicles and trucks.

DENSITY OF MOTOR-VEHICLE TRAFFIC

The traffic map, Figure 1, supports the following general conclusions with respect to the density of traffic:

1. The greatest density of traffic is found on a few main routes entering the county and on the highways adjacent to the city limits of Chicago.
2. The need for improved highways is of markedly less importance in the northwestern and southwestern sections than in the western, northeastern, southeastern, south-central and north-central sections of the county.
3. As distance from the city of Chicago increases the traffic importance of the highways decreases.
4. There are five congested areas in the county, each traversed by one or more highways that carry traffic of greatest density.

On the average day 78,446 motor vehicles, approximately two-thirds of the traffic between the city and county enter or leave the city at the gateways to these five congested areas. There is a most pressing need for additional city connections to diffuse traffic between the city and county, as well as the need for additional through routes in the city. River Road and the recently completed Mannheim Road are the only north-south routes serving the territory west of the city and available for through traffic that wishes to "by-pass" the city. This road passes through thickly populated areas and now carries a heavy traffic that is hampered by intersections with important cross routes. New north-south routes traversing this area as well as east and west routes are needed to diffuse the traffic.

POPULATION AND HIGHWAY TRAFFIC

In connection with the survey an analysis of the distribution of population and the trend of population and industrial development was made. The trends of population growth in Cook County vary markedly in different sections of the county. With some exceptions the trends are similar to the distribution of present population. The areas with the lowest density of population also had the lowest rate of population change between 1910 and 1920. These areas will not produce a large amount of traffic for a considerable time. The west, northeast, and southeast sections of the county, which are adjacent to the city, have a relatively dense population which is increasing rapidly and present the greatest need for highway improvement.

On the basis of past experience highway traffic may be expected to increase even more rapidly than population. The traffic is closely related to the number of traffic units as reflected in motor-vehicle registration, and the registration is increasing more rapidly than the population. In 1914, 31,869 motor vehicles were registered in Chicago, or 1 vehicle for 75.7 persons. In 1924 the registration of motor vehicles was 305,143 or 1 vehicle for 9.64 persons.

FORECAST OF HIGHWAY TRAFFIC IN 1930

The most scientific method of future highway traffic prediction is by the projection of past traffic and motor-vehicle registration trends. In the absence of comprehensive traffic records over a number of years it has been assumed that traffic will increase at the same rate as motor-vehicle registrations, as has been found to be true in Maine, Maryland, Michigan, and Wisconsin.

An additional factor which is negligible in considering a whole State may be of considerable importance in Cook County; that is, the effect of traffic congestion upon the rate of increase of traffic. In Cook County, however, present traffic is affected by congestion, particularly on the highways entering Chicago within 5 miles of the city. Considering the increase in the number of routes planned for construction by 1930, it is doubtful if traffic congestion at that time will be more severe than it is at present, and it may therefore be assumed that the present ratio of traffic to registration will prevail in 1930.

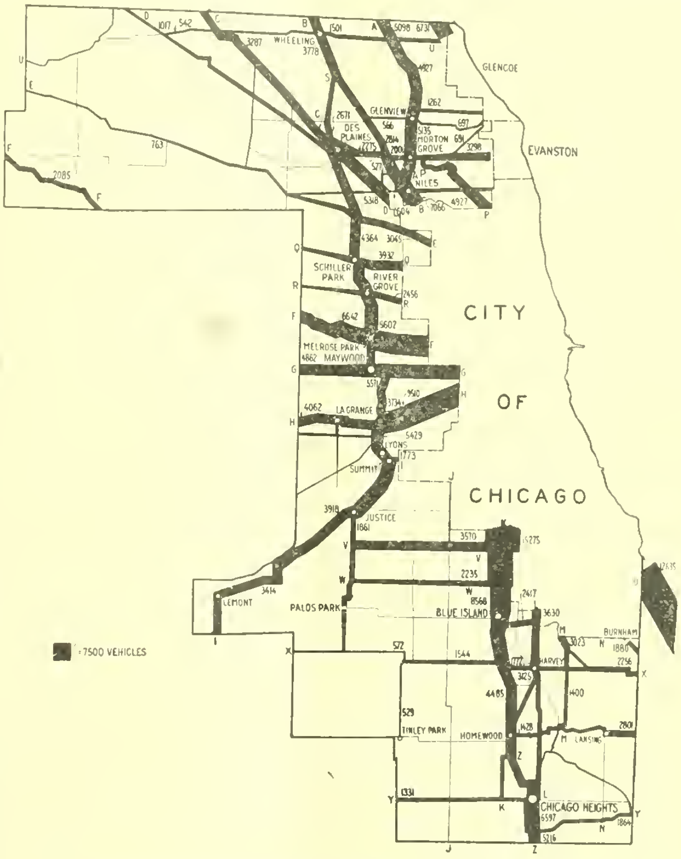

Fig. 1.—Average daily density of vehicular traffic on Cook County highways, 1924

Accurate records of motor-vehicle registration for the entire county of Cook for a series of years are not available. Such records for the city of Chicago are available, and it is necessary, therefore, to assume that the rate of increase of motor-vehicle registration for the county is approximately equal to the rate of increase within the city. The justification of this expedient is evidenced by the fact that in 1920, 88.5 per cent of the total population of the county lived within the city of Chicago. A variation in the rate of registration increase between the city and the county exclusive of the city will therefore have only a small effect upon the rate of increase for the entire county. The fact that population in the county exclusive of the city is increasing more rapidly than the population of the city will tend to make conservative an estimate of future registration based on present city registration.

Vehicles registered in the city of Chicago dominate the traffic on the highways of the county. More than 50 per cent of all trucks recorded on county highways were registered in the city, and trucks registered in the city exceeded by 65 per cent the trucks registered in the county exclusive of the city. It is probable that the distribution of passenger-car registration is quite similar.

The registration of motor vehicles in the city of Chicago from 1914 to 1924 was as follows:

Motor-vehicle registration, by years, in the city of Chicago

Year	Motor-vehicle registration	Year	Motor-vehicle registration
1914	31, 869	1920	109, 609
1915	42, 602	1921	166, 990
1916	60, 072	1922	207, 150
1917	73, 321	1923	259, 043
1918	76, 567	1924	305, 143
1919	93, 125		

An accurate prediction of registration upon the basis of the rate of increase in the past, as shown by these records, will make possible the approximate prediction of future traffic. The predicted percentage increase in registration applied to the present traffic on the highways will give approximate figures for the future total traffic on the highways of the area.

It is not expected that this future traffic will be distributed upon the same routes as at present, owing to the development of new and auxiliary highways, suburban growth, and unusual industrial or pleasure-resort developments. Whether this future traffic can or will be carried upon existing routes or whether new routes are needed may be determined by other considerations. The traffic here predicted for a given route may be divided between the present route and one or more new routes.

Two facts increase the accuracy of such a traffic forecast based on motor-vehicle registration, namely, (1) the fact that comparatively accurate and complete registration statistics are available over a series of years; and (2) the fact that future registration is largely dependent upon future population, which can be accurately predicted for short periods of time.

Future registration is the quotient of future population and future persons per car, i. e., the ratio of population to registration. Admitting the accuracy of the population forecast, we have the problem of forecasting future persons per car.

The table below presents the population of the city of Chicago from 1914 to 1925 as reported by the United States Bureau of the Census, the estimated population from 1926 to 1930 as computed by the method of predicting population used by the Bureau of the Census, and the persons per registered vehicle from 1914 to 1924, the latter determined from the registration statistics in conjunction with the population statistics.

Past population and persons per car in the city of Chicago and estimate of future population

Year	Past population and estimated future population	Persons per car	Year	Past population and estimated future population	Persons per car
1914	2, 412, 218	75. 7	1923	2, 886, 971	11. 1
1915	2, 464, 852	57. 9	1924	2, 942, 605	9. 64
1916	2, 517, 486	41. 9	1925	2, 995, 239	----------
1917	2, 570, 120	35. 1	1926	3, 047, 083	----------
1918	2, 622, 754	34. 3	1927	3, 100, 507	----------
1919	2, 675, 388	28. 7	1928	3, 153, 141	----------
1920	2, 728, 022	24. 9	1929	3, 205. 775	----------
1921	2, 780, 655	16. 7	1930	3, 258, 409	----------
1922	2, 833, 288	13. 7			

From this data it is estimated that there will be 4.86 persons per car in the City of Chicago in 1930. This figure was deduced by plotting years as abscissas and persons per vehicle as ordinates and projecting to 1930 a least square curve fitted to the data from 1914 to 1924. The estimated persons per car for each year to 1930 are shown in the following tabulation:

Estimated persons per car in the city of Chicago from 1925 to 1930

1925	9. 07	1928	6. 14
1926	7. 92	1929	5. 45
1927	6. 96	1930	4. 86

The estimated motor-vehicle registration on the basis of the above figures and the population estimate is as follows:

Estimated motor-vehicle registration in the city of Chicago, 1925 to 1930

Year	Estimated motor vehicle registration	Year	Estimated motor vehicle registration
1925	330, 200	1928	513, 500
1926	384, 700	1929	588, 200
1927	445, 500	1930	670, 500

The 1930 estimated registration of 670,500 vehicles is an increase of 119.7 per cent over the 1924 figure of 305,143 vehicles. This percentage increase applied to 1924 traffic on the Cook County highways gives the estimated traffic for 1930. This estimated traffic is shown graphically in Figure 2.

Figure 2 indicates the probable traffic conditions which will exist on the main highways of Cook County in 1930 unless new routes are added. Western Avenue near the city line may be expected to carry an average of 33,600 vehicles per day. At least five other routes may be expected to carry a daily average in excess of 10,000 vehicles near the city line. If the present ratio between average traffic and Sunday traffic continues until 1930, in excess of 20,000 vehicles may be expected to attempt to use each of these routes on the average Sunday. In view of the present congestion of traffic on these routes, particularly on Sundays, this great increase in traffic will result in impossible traffic conditions.

With few exceptions traffic congestion will, in 1930 as well as at the present time, even if no new routes are added, be limited to a zone approximately 5 miles in width surrounding the city. Five routes may be expected to carry an average daily traffic in excess of 10,000 vehicles throughout their entire length in the county. On the outlying sections of other routes traffic in 1930 is not expected to reach the capacity of the present highway. The northwestern and southwestern sections of the county will continue to be light-traffic areas.

THE PLAN OF IMPROVEMENT

The analysis of traffic upon the present highway system and consideration of the area and population of the area to be served has developed certain fundamental principles which must be incorporated in the plan of highway improvement. These principles may be summarized as follows:

1. The plan must be a regional plan.
2. It must provide for permanent improvements.

3. It must efficiently serve present and expected future traffic. To accomplish this, it is essential that all important sources of highway traffic be connected by direct through routes, involving:

- (*a*) The construction of arterial highways between Chicago and the principal centers of population of the county.
- (*b*) The connection of highways in the area surrounding the city with through city streets, which will permit traffic to move rapidly and directly to its destination.
- (*c*) The construction of routes connecting important cities and villages in the county exclusive of the city of Chicago. These routes will also serve as "by-pass" routes for traffic desiring to avoid the city and as traffic-sorting routes in the present congested areas.
- (*d*) The elimination of obstructions to the free and rapid movement of traffic. This involves the elimination of congestion points such as rail and highway intersections at grade, traffic "bottle necks", and sections of inadequate width.

4. The plan must provide for the acquisition of right of way for required future routes.

5. The present constructed sections of highway must, as far as possible, be utilized as the basis for the proposed plan.

1. *Necessity for a regional plan.*—In order that highway development may adequately meet the actual traffic needs of the Chicago area it will be necessary to coordinate the work of the various agencies now engaged in highway and street improvement in the area. Such coordination can not be accomplished without a comprehensive plan of highway location and construction.

The county of Cook, the city of Chicago, the State of Illinois, and the various smaller political units within Cook County are contemplating large programs of highway and street improvements. Each political unit is limited in its operations by legal restrictions. To obtain, therefore, a unified highway system it is essential that all agencies agree upon a definite improvement plan. By coordinating highway improvements according to an established plan with a known sequence of improvement, each community involved will secure the greatest possible benefit for the least expenditure The plan must serve not only the area as a whole but must also provide improved highway service for each section of the county.

In order to obtain the best results each governmental unit engaged in highway and street improvement should adopt the plan and work in harmony to complete the program of improvement. The poorest section of a highway determines the capacity of the entire route and the failure of one political unit to follow the general plan of improvement may impair highway service to an entire community.

The Chicago City Plan Commission has developed a comprehensive plan of major streets for the city of Chicago. This plan of the county highway system is coordinated with the city plan.

2. *Need for permanent high-type improvements*—The large daily number of passenger cars and motor trucks on the highways of the county requires the best type of design and construction. The plan must therefore provide for highways which will provide efficient service to the large volume of present and expected future traffic. The additional present cost of high-type improvements will be more than equalized by savings in future highway-maintenance costs and motor-vehicle opertaing costs. Improvements must be designed to carry the expected volume of heavy truck traffic as well as the large total number of vehicles.

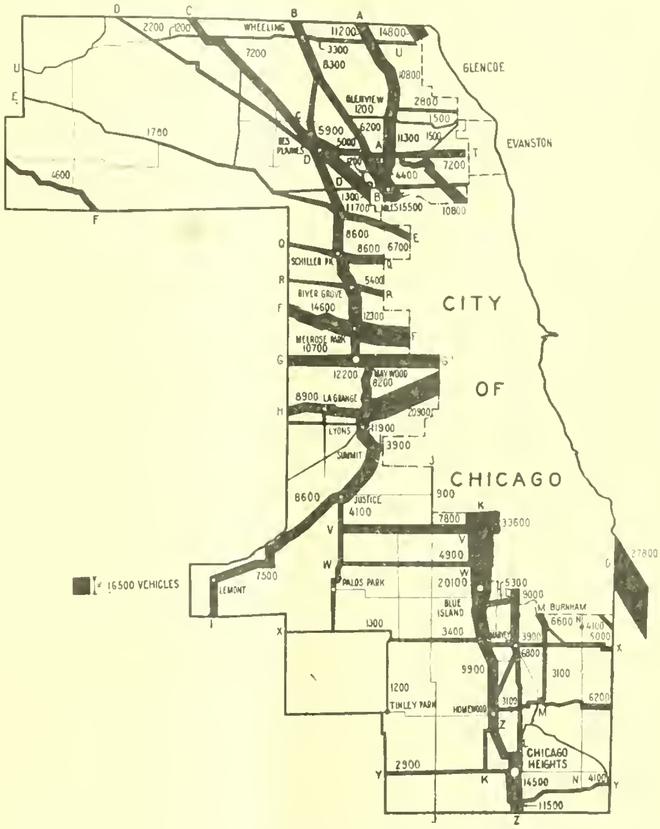

FIG. 2.—Forecast of average daily density of vehicular traffic on Cook County highways, 1930

3. *Need for a highway system which will efficiently serve present and expected future traffic.*—The present congestion of traffic on highways near the city of Chicago is evidence of the inability of the present highway system to serve present traffic. The plan of improvement must provide highway facilities which can carry the present and expected future traffic without the delays and inconveniences caused by present congestion. The analysis of traffic presented in the report indicates the need for the following types of improvements.

a. Arterial highways:

> The distribution of population in Cook County and the principal sources of highway traffic indicate that the greatest need for such arterial highways is within the 5-mile zone surrounding Chicago. The primary movement of traffic on the highway system in the area is between Chicago and the belt of cities and villages surrounding the city. To provide highway service for this movement it is necessary that each of these cities and villages be connected with Chicago by one or more direct routes. A considerable part of the present traffic congestion is caused by the fact that there is at present an inadequate number of such connecting or arterial highways. Traffic from a large suburban area converges at a few points and results in severe congestion on the available routes.

b. The connection of highways in the area surrounding the city with major streets in the city:

The provision of adequate arterial highways to the city line will not eliminate congestion unless these highways are connected with additional through city streets which will permit traffic to move freely and rapidly within the city. If traffic is forced to use a few main thoroughfares within the city, it will continue to "pile up" at the present gateways to the city. The solution of this traffic problem can be found only in the coordinated construction program of the county, the city, and the large number of independent municipalities in the area.

Arterial highways are also required in the sections of the county lying beyond the 5-mile zone adjacent to the city. Present highways will, however, more nearly meet the demands of present and future traffic in this area. The widening of the present routes carrying the heaviest traffic or the provision of a few alternative routes will furnish adequate highway service for a considerable period of years.

c. Circuit and "by-pass" routes:

The rapid growth of the cities and villages in the area surrounding Chicago has greatly increased the local interchange of traffic between these communities. The present highway system does not provide adequate connections between these cities and villages. A large part of the traffic must now use the congested routes leading to the city or must enter the city and use congested city streets. Des Plaines River Road, and the newly completed Mannheim Road are the only north-south circuit routes in the section west of the city, and these routes provide highway facilities for only a limited number of communities. Traffic between Evanston and Oak Park, both cities of over 35,000 population, must either pass through the City of Chicago or must travel almost twice the required distance via Dempster Street and Des Plaines River Road.

The construction of adequate circuit routes would further relieve congestion on the arterial highways by permitting traffic destined to points beyond Chicago to "by-pass" the city, and connections between the arterial and circuit routes would permit the natural sorting of traffic outside the city and allow traffic to enter the city by the most direct route.

d. The elimination of obstructions to the free and rapid movement of traffic:

The required arterial and circuit highways will encounter many of these obstacles and must, therefore, be carefully planned in order that, so far as possible, these barriers may be avoided. In addition to obstructions caused by railway developments and special-use areas the arterial highways radiating from Chicago and the circuit routes around the city must of necessity intersect at a large number of points. Provision must be made for the crossing of these streams of traffic. This will probably involve the separation of grades at certain intersections and the planning of intersections which will involve as little congestion as possible at all intersections.

The traffic capacity of a route is also frequently reduced by narrow sections of pavement, narrow bridges, sharp curves, blind curves, street car and interurban trolley lines, the use of the highway for parking purposes and by local traffic on certain sections of the route, particularly when the route passes through the business section of a community. As far as possible such obstructions to the free movement of vehicles must be eliminated on all important traffic routes.

4. The plan of improvement must provide for the acquisition of right of way for required future use.

The normal development of an area surrounding a large center of population, such as Chicago, results in the establishment of large special-use areas through which highways can not be located. In so far as future highway needs can be predicted, adequate right of way for future improvements should be obtained prior to the establishment of these special-use areas. This procedure will not only reduce the cost of the highway system but will also provide in these areas a highway system so designed as to provide the most efficient highway service. By planning for the future development, many of the obstacles to highway improvement now existing in Cook County can be avoided in the sections of the area which are now undeveloped.

5. The proposed plan shall, as far as possible, utilize the highways already constructed.

Principles of economy dictate that where a through route can be developed by the construction of a relatively small unimproved mileage, this highway should be made a part of the proposed plan, if there is a traffic need for such a highway.

In establishing the order of improvement of the proposed plan it is obvious that improvements should be made in the order of their importance. First consideration should be given to improvements in the areas and on the routes where traffic is seriously congested. Second, the routes necessary to provide service to present traffic should be improved and, third, the highway right of way should be obtained on routes planned to serve future traffic.

In accordance with these principles a detailed plan of highway improvement has been worked out which includes the construction of pavements on approximately 838 miles of highway, of which approximately 248 miles must be opened as well as paved, and the widening of approximately 97 miles of present pavement. This mileage has been classified in four stages according to the urgency of the needed improvement.

The completion of the proposed plan of highway improvement will provide for Cook County, for the city of Chicago, and for the surrounding area a highway system which will serve the large volume of present highway traffic in the area and the expected increase of traffic in the immediate future.

The translation of the plan into actual improvements is already under way. This fact is evidenced by the construction during 1925 of a considerable mileage of urgently needed improvements in the county. Legislation enabling the cooperating agencies to carry out the proposed plan in the most efficient manner is enacted or pending.

Working agreements between the principal agencies charged with the development of the highway system have already been established. Agreements with municipalities, required for the initiation of improvement projects on certain routes, are rapidly being established and the details of a coordinated construction program are being arranged.

The execution of the proposed plan will require a considerable outlay of public funds. The plan is so arranged, both as to routes to be improved and the order of their improvement, that these funds will be placed where the greatest return on the investment will be realized. The required outlay of funds will be quickly returned in the form of reduced transportation costs, increased real-estate values, and improved highway service.

The severity of present traffic congestion in the area and the rapid increase in highway traffic, population, and industrial development in Cook County demonstrate the need for continuous and intensive study of the problem of providing highway service in order that present conditions may not recur.

EFFECTIVE WIDTH OF CONCRETE BRIDGE SLABS SUPPORTING CONCENTRATED LOADS

Reported by **E. F. KELLEY**, Acting Chief, Division of Tests, United States Bureau of Public Roads

IN THE DESIGN of a narrow rectangular reinforced concrete beam, subjected to concentrated loads, it is permissible to assume that the stress is constant over the full width of the beam. This assumption can not be made in the case of a wide slab where the stress varies from a maximum at the point of application of the load to a minimum at the edge of the slab. Therefore, in order to apply the rectangular beam theory to the design of wide slabs it becomes necessary to determine the width of slab, commonly called the "effective width," over which the concentrated load may be assumed as uniformly distributed.

It is the purpose of this paper to consider the data on the subject which are available and to derive working rules for distribution, suitable for use in the design of the floor slabs of highway bridges. The results of three series of tests on stress distribution have been published and these furnish the fundamental data for this study. The tests referred to are those of the University of Illinois, the State Highway Department of Ohio, and the United States Bureau of Public Roads. The conclusions based on these tests will be presented briefly in the following pages, but for complete discussions the reader is referred to the original papers.[1]

CONCLUSIONS FROM BUREAU OF PUBLIC ROADS TESTS

Of the several tests those by the Bureau of Public Roads are the most comprehensive, and these are summarized in the article published in the issue of Public Roads for September, 1918. The conclusions drawn from them may be briefly stated as follows:

Influence of slab width on effective width.—If the slab width be designated as b, the span of the slab as S, and the effective width as e, the relation between $\frac{b}{S}$ and $\frac{e}{S}$ which has been determined for a slab centrally loaded with a single load is shown in Figure 1.

Influence of two-point loading on effective width.—From tests made with two loads 5 feet apart on the center line of the slab, a loading approximating that of a single-axle load of a truck, it is concluded that the effective width equals the effective width for a single central load plus 4 feet.

Influence of eccentricity of load.—When a slab is eccentrically loaded with a single load, the effective width is the same as for a centrally loaded slab, when the distance, D, of the load from the near edge of the slab is more than one-half the effective width of the centrally loaded slab. When the distance D is less than half the effective width under a central load, the effective width equals $\frac{e}{2}+D$ where e is the effective width under a central load.

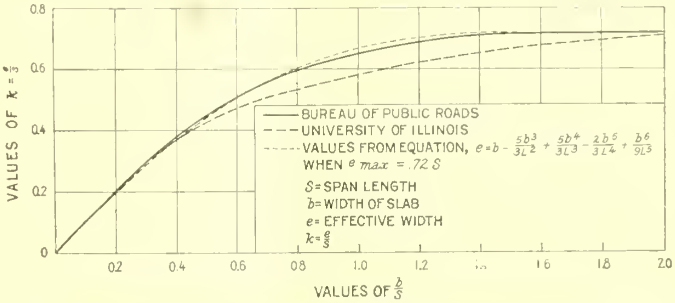

FIG. 1.—Relation between slab width and effective width as determined by the Bureau of Public Roads and the University of Illinois

UNIVERSITY OF ILLINOIS CONCLUSIONS

The results of the University of Illinois tests are given by Mr. Slater in his discussion of the earliest tests by the Bureau of Public Roads. He presents a graph showing the relation between slab width and effective width which is shown, in comparison with the same relation as established by the bureau, in Figure 1. It is apparent that the effective widths as proposed by Mr. Slater are in fairly close agreement with those proposed by the bureau, although somewhat lower. But it should be noted that both determinations of the effective width are for a load applied at a point. To provide for the fact that actual loads, such as truck wheels, have an appreciable width of bearing, Mr. Slater proposed the following rule:

$$E=2x\tan\theta+w$$

where E = effective width for load of width w
x = distance from load to near support
e = effective width for load applied at a point
S = span length

$$\tan\theta=k=\frac{e}{S}\text{ (values given by curves, Figure 1).}$$

When, therefore, $x=\frac{S}{2}$, the equation becomes

$$E=kS+w$$

In his very interesting article in the Engineering News Professor Young carries Mr. Slater's proposals somewhat further and develops equations for effective widths (fig. 2) which take into account the width and length of wheel contact area and also the direction of traffic with reference to the direction of the span of the slab. When k is substituted for $\tan\theta$ in these equations they become, for Cases I and II, respectively:

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[1] *Tests of the State Highway Department of Ohio:*

Load Distribution Tests of Reinforced Concrete Slab Floors under Concentrated Loads. Bull. No. 28, State Highway Department of Ohio.

Tests of the U. S. Bureau of Public Roads:

Tests of Reinforced Concrete Slabs under Concentrated Loading. By A. T. Goldbeck. Proceedings of the American Society for Testing Materials (1913), vol. 13, p. 858–873.

Test of a Reinforced Concrete Slab. By E. B. McCormick. Journal of American Concrete Institute, May, 1915, vol. 3, No. 5, p. 195–204.

Tests of Large Reinforced Concrete Slabs. By A. T. Goldbeck and E. B. Smith. Proceedings of the American Concrete Institute (1916), vol. 12, p. 324–333.

Tests of Three Large-Sized Reinforced Concrete Slabs under Concentrated Loading. By A. T. Goldbeck and E. B. Smith. Jour. Agr. Research, Vol. VI, No. 6, May 8, 1916.

The Influence of Total Width on the Effective Width of Reinforced Concrete Slabs Subjected to Central Concentrated Loading. By A. T. Goldbeck. Proceedings of the American Concrete Institute (1917), vol. 13, p. 78–88.

Tests of a Large-Sized Reinforced Concrete Slab Subjected to Eccentric Concentrated Loads. By A. T. Goldbeck and H. S. Fairbank. Jour. Agr. Research, Vol. XI, No. 10, Dec. 3, 1917.

Reinforced Concrete Slab Bridge Design Based on Full-Sized Tests. By A. T. Goldbeck. Public Roads, Vol. 1, No. 5, September, 1918.

Tests of the University of Illinois.

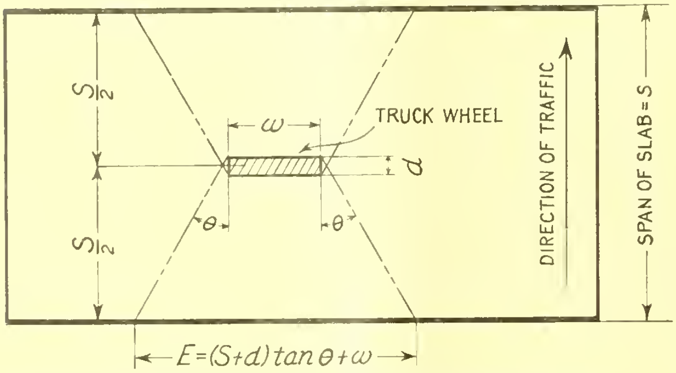
Discussion of Mr. Goldbeck's paper on Tests of Reinforced Concrete Slabs under Concentrated Loading. By W. A. Slater. Proceedings of the American Society for Testing Materials (1913), vol. 13, p. 874–883.

The Effective Width of Reinforced Concrete Slabs Supporting Concentrated Loads. By C. R. Young. Engineering News, July 30, 1914.

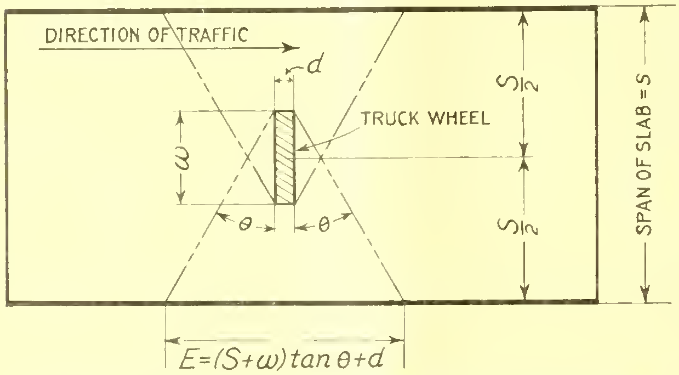
$$E=k(S+d)+w$$
$$\text{and } E=k(S+w)+d$$

and as the dimension, d, is small as compared with the width of wheel, it is advantageous, from the standpoint of simplicity, to neglect it, in which case the above equations reduce to:

$$E=kS+w \qquad \text{(Case I)}$$
$$E=k(S+w) \qquad \text{(Case II)}$$



CASE I DIRECTION OF TRAFFIC SAME AS SPAN OF SLAB



CASE II DIRECTION OF TRAFFIC PERPENDICULAR TO SPAN OF SLAB

FIG. 2.—Diagram of area of wheel contact in relation to slab

The fundamental principles underlying the determination of effective width are well stated in the paper by Goldbeck and Smith,[2] from which the following is quoted:

> The width of the slab that should be used in the rectangular-beam formulas when applied to slab design will be termed the "effective width" of the slab. It is that width over which, if the stress were constant and equal to the maximum stress under actual conditions, the resisting moment would equal the resisting moment of a slab of the same depth and full width, but having varying stress distribution. If the straight-line theory of stress distribution from neutral axis to upper fibers is assumed to be applicable to slabs, the resisting moment of a given slab is dependent on the total stress in the concrete or steel at the dangerous section. The total stress in the concrete, however, is governed by the stresses in the top fibers, and these stresses are proportional to the unit deformations. If, then, there are two slabs of equal depth, one having uniform distribution of deformations and the other a varying distribution, but with their maximum deformations identical, they will likewise have equal resisting moments if the summations of the deformations over their respective widths are identical.

In Figure 3, which represents a slab in position on two supports with a concentrated load P, is illustrated the method of obtaining "effective width." Strain-gauge readings are taken of the fiber deformations perpendicular to the supports, as indicated at eg. These concrete deformation values are plotted to scale, as, for instance, at fh, giving the deformation curve JHF, inclosing the area $AJHFE$. This curve shows the variation of stress from the center to each of the two free edges of the slab, and the area under the curve is a function of the total concrete-resisting moment of the slab. The area $BDGI$ (formed upon the base BD),[3] obtained by dividing the area $AJHFE$ by its maximum ordinate CH, has the same total concrete-resisting moment with the stress uniformly distributed as the whole slab, and its width BD, is that which may be effective in furnishing sufficient resistance under these conditions to carry the load. The width BD, obtained in this manner, is the "effective width."

CONCLUSIONS FROM THE OHIO TESTS

The results of the Ohio tests, in so far as they may be compared, are in close agreement with those of the University of Illinois and the Bureau of Public Roads, but the influence of slab width was not determined and, since the results may be applied only to wide slabs, their application to bridge design is limited.

However, in the published results (Bul. 28), an application of theoretical principles is developed which is worthy of careful study and which seems to the writer to offer opportunities for more extended consideration than has yet been given it.

The following is quoted from the above-mentioned bulletin:

> From the results of the tests it is known that the intensity of the resistance decreases from the load out, and that the shape of the transverse deformation curve is similar to that of the elastic line of a beam fixed at the ends. We may assume then that the elastic curve of a beam fixed at the ends and loaded as shown in Figure 4, would approach the form of the actual transverse deformation curves derived from the tests.

M_1 = moment necessary to hold the beam horioztnal at the ends.
w = maximum intensity of a reaction varying uniformly to zero at the ends.
y = deflection at any point.
W = total load $=\frac{wL}{2}$, where L is the span of the transverse deformation curve.

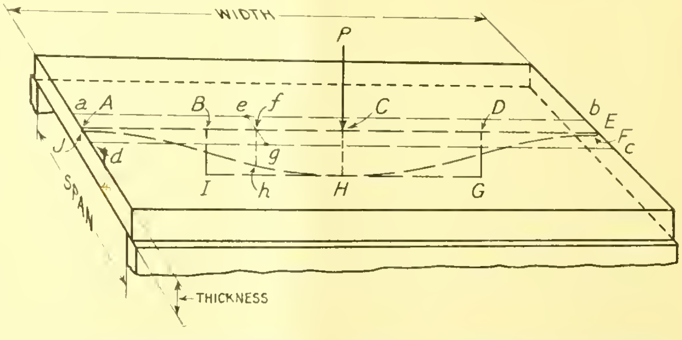


FIG. 3.—Diagram illustrating Bureau of Public Roads method of determining the effective width of a slab

The equation of the elastic line is:

$$y=\frac{W}{6EI}\left[\frac{x^5}{5L^2}-\frac{Lx^2}{16}\right] \qquad (1)$$

$$\text{When } x=\frac{L}{2},\ y=y_{\max.}=-\frac{WL^3}{640EI} \qquad (2)$$

Let b = the width of slab where b is less than L.

Then the area under the elastic line for the length b will correspond to the areas under the actual transverse deformation curves.

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[2] Tests of Three Large-Sized Reinforced Concrete Slabs under Concentrated Loading. By A. T Goldbeck and E B. Smith Jour. Agr. Research, Vol. VI, No. 6, May 8, 1916.

[3] Inserted by the editor.

$$A=2\int_{\frac{L-b}{2}}^{\frac{L}{2}} y\,dx=\frac{W}{3EI}\int_{\frac{L-b}{2}}^{\frac{L}{2}}\left[\frac{x^5}{5L^2}-\frac{Lx^2}{16}\right]dx$$

$$=\frac{W}{5760EI}\left[-9L^3b+15Lb^3-15b^4+\frac{6b^5}{L}-\frac{b^6}{L^2}\right] \qquad (3)$$

$$e=\text{effective width}=\frac{A}{y_{\text{max.}}}=b-\frac{5b^3}{3L^2}+\frac{5b^4}{3L^3}-\frac{2b^5}{3L^4}+\frac{b^6}{9L^5} \qquad (4)$$

when $b=L$, e will be a maximum, or the effective width of a slab of indefinite width,

$$e_{\text{max.}}=\frac{4}{9}L \qquad (5)$$

* * * * *

No tests were made with two loads on the slab at the same time, but their effect may be studied by the use of the deformation curve given by equation 1.

* * * * *

The curve for each load may be assumed to be independent of the other and the total deformation at any point may be taken as the sum of the two deformations at that point.

APPLICATION OF TEST RESULTS TO BRIDGE DESIGN

In the above series of tests certain limitations are noted which affect the application of the results to actual bridge design, as follows:

1. The influence of slab width for *single central* loads is quite definitely established by the tests of the Bureau of Public Roads and University of Illinois. The Ohio tests give results only for wide slabs.
2. For loads applied at two points on the same transverse element of the slab, the only data are those furnished by the Bureau of Public Roads tests. These tests were only for loads spaced 5 feet apart and therefore do not give results which are applicable to other spacings of loads.

The theoretical principles developed in Ohio Bulletin No. 28 were applied only to two loads on wide slabs.

We are therefore still in considerable doubt as to the effect of two wheels when both wheel spacing and width of slab are variables and, as yet, there are no data on the effect of four loads, such as would be occasioned by two trucks passing.

3. The effect of eccentric loads has been studied only by the Bureau of Public Roads and the results of the tests made are not entirely conclusive, as is evidenced from the following, quoted from the paper by Goldbeck and Fairbank.[4]

> Unfortunately, however, the results can not be regarded as entirely conclusive, owing to the fact that the recent test involved only one span length and yielded only two points on the critical part of the curve between the total widths of 18 and 22 feet. It is hoped that this point will be cleared up by further tests.

In view of the above it is highly desirable to investigate more fully the various conditions of loading which may arise and, in the absence of test data, it is believed that this may be done satisfactorily by developing further the theory proposed in Ohio Bulletin No. 28.

Referring, then, to this theory and substituting values of $\frac{b}{L}$ in equation 4, we obtain the values of $\frac{e}{L}$ given in the second column of Table 1.

But from equation 5 we have $e_{\text{max.}}=\frac{4}{9}L$ and from the Bureau of Public Roads curve, which may be assumed to give conservative values, we have $e_{\text{max.}}=0.72S$

Equating these values of $e_{\text{max.}}$

$$L=1.62S \qquad (6)$$

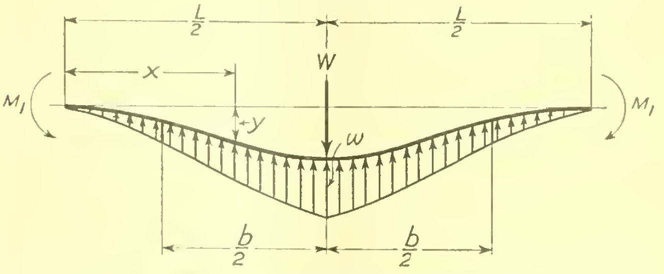

FIG. 4.—Elastic curve and loading diagram of beam with fixed ends used in development of theory based on the Ohio tests

By substituting this value of L in the values of $\frac{b}{L}$ and $\frac{e}{L}$ given in the first two columns of Table 1 we obtain the values of $\frac{b}{S}$ and $\frac{e}{S}$ given in the third and fourth columns of Table 1 and these have been plotted in Figure 1.

The close agreement of the theoretical curve with that proposed by the bureau is very striking and there is so little difference between the two that, for all practical purposes, they may be said to be the same. The values proposed by the bureau and those given by equation 4 are compared for identical values of $\frac{b}{S}$ in the last two columns of Table 1.

TABLE 1.—*Values of* $\frac{e}{L}$, $\frac{b}{L}$, $\frac{e}{S}$, *and* $\frac{b}{S}$, *derived from equation 4 and the Bureau of Public Roads curve*

Values of $\frac{e}{L}$ computed from equation 4 with assumed values of $\frac{b}{L}$		Values of $\frac{b}{S}$ and $\frac{e}{S}$ computed from equations 4 and 6		Comparison of values of $\frac{e}{S}$, corresponding to assumed values of $\frac{b}{S}$, as given by equation 4 and the Bureau of Public Roads curve		
					$\frac{e}{S}$	
$\frac{b}{L}$	$\frac{e}{L}$	$\frac{b}{S}$	$\frac{e}{S}$	$\frac{b}{S}$	Bureau of Public Roads curve	Equation 4, $e_{\text{max.}}=.72S$
0.05	0.0498	0.081	0.081	0.1	0.1	0.1
.10	.0985	.162	.160	0.2	0.2	0.2
.15	.1452	.243	.235	0.3	0.28	0.28
.20	.1891	.324	.306	0.4	0.37	0.37
.25	.2298	.405	.372	0.5	0.44	0.44
.30	.2670	.486	.433	0.6	0.50	0.51
.35	.3003	.567	.486	0.7	0.55	0.56
.40	.3296	.648	.534	0.8	0.58	0.61
.45	.3551	.729	.575	0.9	0.62	0.64
.50	.3767	.810	.610	1.0	0.65	0.67
.55	.3947	.891	.639	1.1	0.67	0.69
.60	.4093	.972	.663	1.2	0.68	0.70
.65	.4208	1.053	.682	1.3	0.70	0.71
.70	.4295	1.134	.696	1.4	0.71	0.72
.75	.4358	1.215	.706	(1)	0.72	0.72
.80	.4400	1.296	.713	----------	----------	----------
.85	.4426	1.377	.717	----------	----------	----------
.90	.4439	1.458	.719	----------	----------	----------
.95	.4444	1.539	.720	----------	----------	----------
1.00	.4444	1.620	.720	----------	----------	----------

[1] 1.5 and over.

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[4] Tests of a Large-Sized Reinforced Concrete Slab Subjected to Eccentric Concentrated Loads. By A. T. Goldbeck and H. S. Fairbank. Jour. Agri. Research, Vol. XI, No. 10, Dec. 3, 1917

This agreement between the test curve and the theoretical curve indicates that values of L may be taken as independent of the width of slab and dependent only on the span length. Accepting the maximum value for effective width as proposed by the Bureau of Public Roads, the relation between L and the span length is given by equation 6.

Having thus determined the value of L, the span of the transverse deformation curve, we may proceed to the consideration of multiple loads.

In Figure 5 is shown a slab of width, b, loaded with a single concentrated load. The curve ABC represents the transverse deformation curve of length, L, and maximum ordinate, y. The effective width equals the area under the curve for the length, b, divided by the maximum ordinate, y; that is, the area $DEFBG$ divided by y.

In Figure 6 the slab of width, b, is shown loaded with two equal concentrated loads P_1 and P_2 and with transverse deformation curves ABC and DEF, respectively. The curve AGF is the summation curve of the two curves ABC and DEF and therefore represents the deformation curve for the two loads. The effective width for the two loads is therefore the area $HJGKL$ divided by the maximum ordinate, y_m.

It will be noted that the maximum ordinates y_1 and y_2 of the two deformation curves are not equal. This is due to the fact that the corresponding effective widths for the two loads are not equal, as shown by the following:

Let A_1 = area $HJMBL$

A_2 = area $DENL$

e_1 = effective width for load P_1

e_2 = effective width for load P_2

Then $e_1 = \frac{A_1}{y_1}$ and $e_2 = \frac{A_2}{y_2}$.

As previously stated, the area under the deformation curve is a function of the total resisting moment of the slab. Since the loads P_1 and P_2 are equal, the respective resisting moments of the slab must also be equal and therefore the areas under the two deformation curves must be equal.

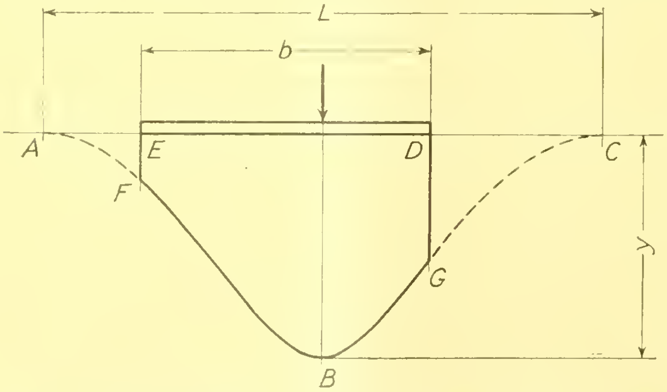


FIG. 5.—Transverse deformation curve of a slab with single concentrated eccentric load

The $A_1 = A_2$ and, from the above two equations,

$$\frac{y_2}{y_1} = \frac{e_1}{e_2}.$$

Thus it is clear that when the effective widths are unequal the corresponding maximum ordinates of the deformation curves are also unequal.

DETERMINATION OF THE EFFECTIVE WIDTH

Knowing the characteristics of the deformation curve, the principles outlined above may be applied to any combination of loads on slabs of varying spans and widths, and the resulting effective widths determined. The method is illustrated by the two typical examples which follow:

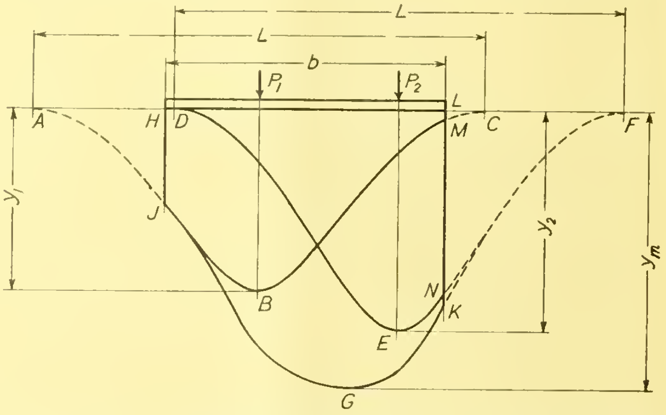


FIG. 6.—Transverse deformation curve of a slab with two equal concentrated loads unsymmetrically placed

EXAMPLE I

Assume a slab of 20-foot span and 20-foot width loaded at the center of the span with two equal loads P_1 and P_2 which are placed transversely in the positions shown in Figure 7 (a).

From equation 6,

$$L = 1.62S = 1.62 \times 20 = 32.4 \text{ feet.}$$

Using the value of L thus determined and the known width of the slab, b, the effective width, e, may be obtained from equation 4, or from Figure 8, which presents the relation between $\frac{b}{L}$ and $\frac{e}{L}$ expressed by the equation. The value of e thus obtained, however, will be for a single load centrally placed. In order to determine the effective width for each load eccentrically placed as in Figure 7 (a) the following method may be used.

Figure 7 (b) shows the transverse deformation curve for a slab of width $(a+c)$ unsymmetrically loaded with a single concentrated load. Since the deformation curve is symmetrical about the load it is evident that the area $ABCDE$ equals twice the area $ABCK$, and the area $FGCHJ$ equals twice the area $KCHJ$.

Let A = area $ABCHJ$

A_a = area $ABCDE$

A_c = area $FGCHJ$

Then $A = \frac{1}{2}(A_a + A_c)$. (7)

But the effective width is equal to the area under the deformation curve divided by the maximum ordinate.

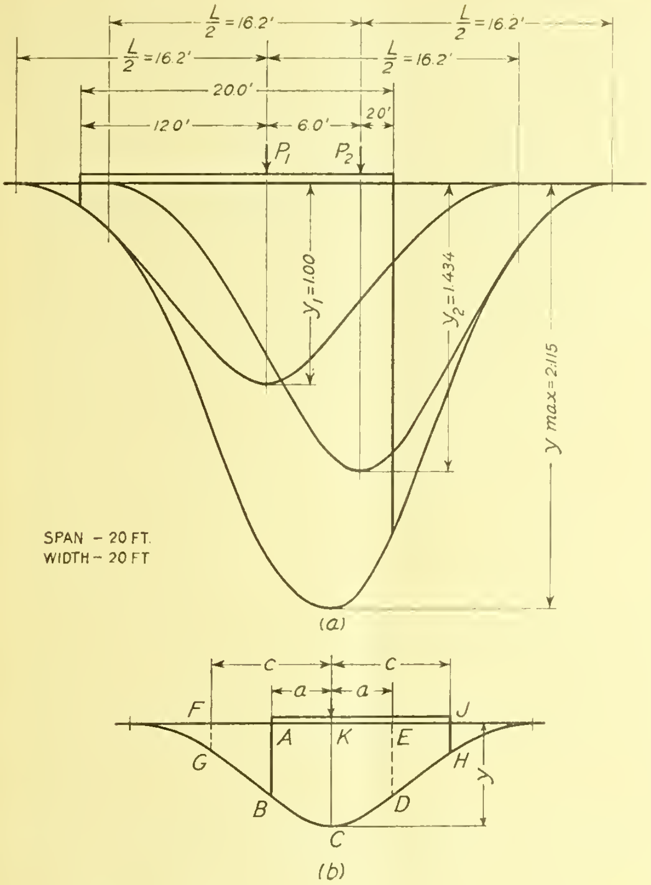


FIG. 7. (a) Diagram of loading of 20-foot slab, 20 feet wide and transverse deformation curves resulting from eccentric loads applied at center of span. (b) Diagram of slab of width ($a+c$) loaded unsymmetrically with a single concentrated load, and deformation curve resulting from load

Therefore, $e=\frac{A}{y}=$ effective width for the slab as shown.

$e_a=\frac{A_a}{y}=$ effective width for a centrally loaded slab of width $2a$.

$e_c=\frac{A_c}{y}=$ effective width for a centrally loaded slab of width $2c$.

And substituting in equation 7, $e=\frac{1}{2}(e_a+e_c)$.

From Figure 8, e_a may be obtained by taking b as equal to $2a$ and e_c may be obtained by taking b as equal to $2c$.

For the load P_1, in the example under consideration the effective width, e_1, is equal to $\frac{1}{2}(e_a+e_c)$,

where $e_a=$ effective width for a centrally loaded slab of width $b_a=2\times 12=24$ feet, and
$e_c=$ effective width for a centrally loaded slab of width $b_c=2\times 8=16$ feet.

For this load then, the curve, Figure 8, gives the following values:

$$\text{For } \frac{b_a}{L}=\frac{24}{32.4}=0.7407;\ \frac{e_a}{L}=0.435$$

$$\text{For } \frac{b_c}{L}=\frac{16}{32.4}=0.4938;\ \frac{e_c}{L}=0.374$$

Then $e_1=\frac{1}{2}(0.435L+0.374L)=0.4045L$.

Similarly, for the load P_2 the effective width, e_2, is equal to $\frac{1}{2}(e_a+e_c)$

where $e_a=$ effective width for a centrally loaded slab of width
$b_a=2\times 18=36$ feet and
$e_c=$ effective width for a centrally loaded slab of width
$b_c=2\times 2=4$ feet
$e_a=0.444L$ and $e_c=0.120L$
and $e_2=\frac{1}{2}(0.444L+0.120L)=0.282L$.

Then, from the previous discussion $\frac{y_2}{y_1}=\frac{e_1}{e_2}$

$$\text{If } y_1=1.\ y_2=\frac{0.4045L}{0.282L}=1.434$$

Using these values of the maximum ordinates, the deformation curves for the loads P_1 and P_2 may now be drawn, as shown in Figure 7 (a), the lesser ordinates being determined from equation 1 or from Table 2 which has been constructed from this equation assuming the maximum ordinate to be unity. The curves for the two loads being constructed in this way, the summation curve may be drawn and its maximum ordinate is found to be 2.115.

The area of the summation curve equals the sum of the respective areas A_1 and A_2 of the curves for the loads P_1 and P_2. But as it has already been shown that A_1 equals A_2, the area, A, of the summation curve is equal to $2A_1$.

$$\text{Then since } e_1=0.4045L=\frac{A_1}{y_1}$$

and since $y_1=1.\ A_1=0.4045L$
and $A=2\times 0.4045L$.

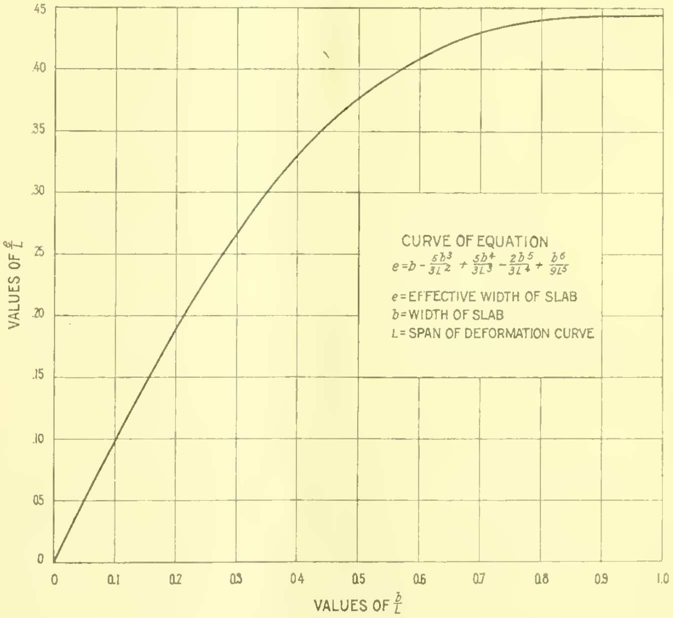


FIG. 8.—Relation of $\frac{e}{L}$ and $\frac{b}{L}$ as expressed by Ohio equation

And, finally, if we let $E=$ effective width for the two loads, and

$e=\frac{E}{2}=$ effective width for one of the loads

$$\text{Then } E=\frac{A}{y_{\text{max.}}}=\frac{2A_1}{y_{\text{max.}}},\ \text{and}$$

$$e=\frac{2\times 0.4045L}{2\times 2.115}=\frac{0.4045\times 32.4}{2.115}=6.20\text{ feet}$$

TABLE 2.—*Coordinates of deformation curve when* $y_{\max}=1$ *derived from equation 1,* $y=\frac{W}{6EI}\left[\frac{x^5}{5L^2}-\frac{Lx^2}{16}\right]$

Values of x	Values of y	Values of x	Values of y
0.05L	0.017	0.35L	0.702
.10L	.066	.40L	.848
.15L	.149	.425L	.908
.20L	.260	.45L	.957
.25L	.396	.475L	.988
.30L	.548	.50L	1.000

EXAMPLE II

Assume a slab of 22-foot span and 24-foot width loaded at the center of the span with four equal loads P_1, P_2, P_3, and P_4 as shown in Figure 9.

$$L=1.62S=1.62 \times 22=35.64 \text{ feet}$$

By the method described in Example I the following values of effective widths for the various loads are found:

$$e_1=0.385L$$
$$e_2=0.424L$$
$$e_3=0.399L$$
$$e_4=0.277L$$

And if, as before, $y_2=1$

$$y_1=1.101$$
$$y_3=1.062$$
$$y_4=1.531$$

The deformation curves with the above maximum ordinates may then be drawn and from them the summation curve constructed; and, as from the previous discussion it is known that the areas under the various deformation curves must be equal, that is, $A_1=A_2=A_3=A_4$.

Therefore, A (area of summation curve) $=4A_2$.

But, since $y_2=1$, $A_2=e_2=.424L$ and from the summation curve, $y_{\max}=3.42$.

If, again, we let

E = effective width for the four loads, and

$e=\frac{E}{4}$ = effective width for one of the loads.

Then $E=\frac{A}{y_{\max}}=\frac{4A_2}{y_{\max}}$, and

$$e=\frac{4\times0.424L}{4\times3.42}=\frac{0.424\times35.64}{3.42}=4.42 \text{ feet.}$$

EFFECTIVE WIDTH OF SLABS WITH MAIN REINFORCING PARALLEL TO DIRECTION OF TRAFFIC

Following the method outlined above, effective widths have been determined for various conditions of truck loading on slabs from 12 to 36 feet in width and of spans from 2 to 24 feet; and the results are tabulated in Tables 3 and 4 for four conditions of loading designated as Cases I, II, III, and IV. In some cases an approximate method for the determination of the maximum ordinate of the summation curve has been used in order to eliminate a great amount of tedious curve construction, and this has caused slight errors in the results. However, these errors do not exceed 2 or 3 per cent and are on the side of safety.

The cases considered include slabs symmetrically and eccentrically loaded with one and two trucks. For eccentric loading the minimum distance from the center of the outer wheel to the edge of the slab has been taken as 2 feet, which is a fair minimum figure for any slab carrying a curb or rail.

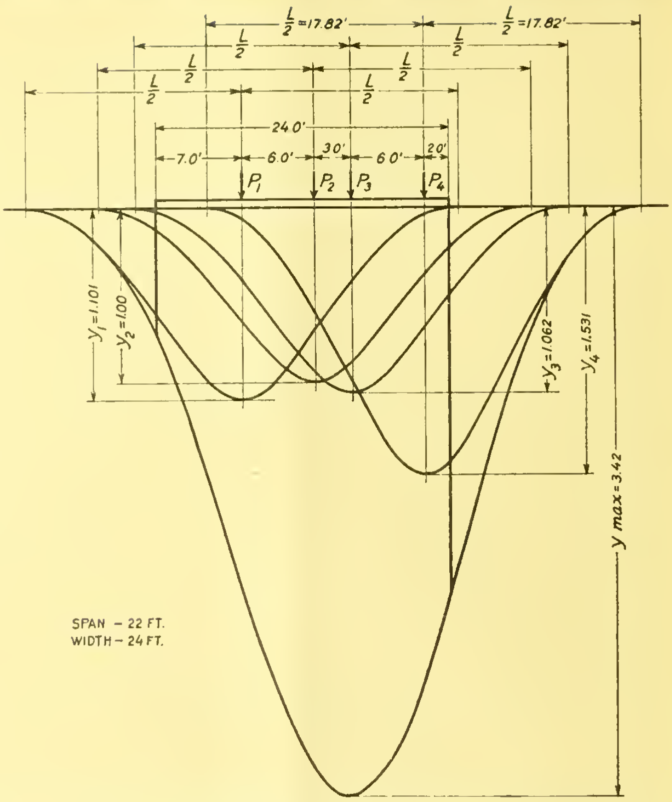


FIG. 9.—Diagram of loading of 22-foot span, 24 feet wide loaded at the center of the span with four equal loads, and transverse deformation curves resulting from loads

It will be noted that the values of effective width for Case II are equal to or greater than the corresponding values for Case I, and that the values for Case IV, except for a few slight discrepancies probably due to unavoidable errors in computation, are equal to or greater than the values for Case III. Since a conservative specification must provide for the worst conditions—in this case, the minimum effective widths—we may eliminate Cases II and IV and consider only Cases I and III in deriving approximate values for distribution which may be used in slab design.

The curves in Figure 10 show the maximum and minimum values of effective width for Cases I and III, as given in Table 3. For a single concentrated load the effective width has been shown to be greatly influenced by the slab width, but these curves show clearly that, for the multiple loads here considered, the effective width, except for the slab of 12-foot width in Case I, is influenced very little by the slab width and depends almost entirely on the span of the slab.

The straight dotted lines on the diagrams represent approximate values of effective width which agree closely with the computed values and are convenient for design purposes.

The equations of the dotted lines are as follows:

For Case I (one truck):

$e=0.65S$, for spans of 7 feet or less;

$e=0.12S+3.71$, for spans of more than 7 feet.

For Case III (two trucks):

$e=0.65S$, for spans of 4 feet or less;

$e=0.25S+1.6$, with a maximum value of 4.4, for spans of more than 4 feet.

It should be noted that Cases I and III involve slabs in which the main reinforcement is in the direction of the traffic; also that, in computing the effective widths, no account has been taken of the width of wheel contact. Professor Young's equation, $E=kS+w$ takes into account the width of wheel contact when the main reinforcement is in the direction of the traffic and we may therefore substitute for kS in this equation the values of the effective widths as determined for Cases I and III above. Making these substitutions we obtain the following equations for the effective width of a single wheel of a group when the main reinforcement of the slab is parallel to the direction of the traffic.

For one truck:

For spans of 7 feet or less, $e=0.65S+w$.

For spans of more than 7 feet, $e=0.12S+w+3.71$.

For two trucks:

For spans of 4 feet or less, $e=0.65S+w$.

For spans of more than 4 feet, $e=0.25S+w+1.6$, with a maximum value of $4.4+w$.

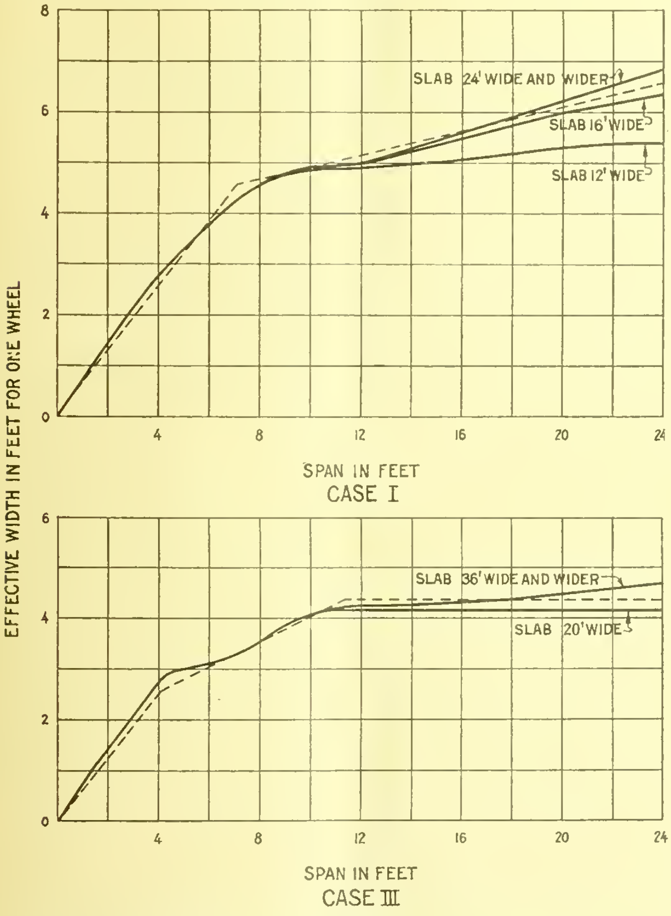


FIG. 10.—Maximum and minimum values of effective width for one wheel (eccentric loading), the main reinforcing at the slab being parallel to the direction of the traffic, and the effect of tire width neglected

Referring again to the curves of Case I, Figure 10, it may seem at a glance that the approximate equation represented by the dotted line is not in sufficiently close agreement with the values for the slab of 12-foot width. However, it can be shown that this is not the case when the width of wheel contact and the fact that the sum of the effective widths for all the loads on the slab can not exceed the width of the slab are taken into consideration. Since this slab of 12-foot width carries two wheel loads, the effective width for one of them can not exceed 6 feet, and it will be reasonable to assume the width of wheel as 1 foot. Substituting these values in the equation for effective width we find that, for a wheel 1 foot wide, the effective width is 6 feet when the span length is 10.75 feet. For spans less than 10.75 feet, the actual values for the 12-foot slab width are shown by Figure 10 to be in close agreement with those of the approximate equation while, for spans greater than 10.75 feet the equation does not apply since the effective width is limited by the width of slab. Therefore, the actual values for the 12-foot slab width are in practically as close agreement with those of the approximate equation as are the actual values for the other widths of slab.

TABLE 3.—*Effective width for one wheel (eccentric loading). Main reinforcement parallel to direction of traffic; effect of tire width neglected*

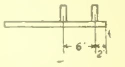
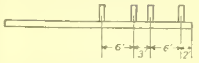
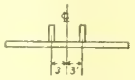
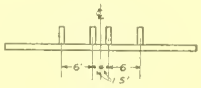
Condition of loading	 Case I					 Case III			
	Width of slab					Width of slab			
Span (feet)	12	16	18	20	24 and wider	20	24	30	36 and wider
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
2	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44
4	2.78	2.78	2.78	2.78	2.78	2.78	2.78	2.78	2.78
6	3.79	3.79	3.79	3.79	3.79	3.15	3.15	3.15	3.15
8	4.61	4.61	4.61	4.61	4.61	3.59	3.59	3.59	3.59
10	4.86	4.93	4.93	4.93	4.93	4.07	4.07	4.07	4.07
12	4.85	5.01	5.01	5.01	5.01	4.25	4.29	4.29	4.29
14	4.98	5.25	5.30	5.30	5.30	4.23	4.32	4.34	4.34
16	5.12	5.53	5.59	5.60	5.60	4.19	4.34	4.38	4.38
18	5.20	5.76	5.84	5.88	5.88	4.16	4.38	4.42	4.42
20	5.30	6.00	6.12	6.20	6.22	4.17	4.43	4.51	4.52
22	5.39	6.20	6.41	6.50	6.55	4.18	4.42	4.59	4.60
24	5.40	6.37	6.61	6.75	6.85	4.21	4.55	4.71	4.73

TABLE 4.—*Effective width for one wheel (symmetrical loading). Main reinforcement parallel to direction of traffic; effect of the tire width neglected*

Condition of loading	 Case II							 Case IV			
	Width of slab							Width of slab			
Span (feet)	12	16	18	20	24	30	36	20	24	30	36
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
2	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44
4	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88
6	4.17	4.32	4.32	4.32	4.32	4.32	4.32	3.15	3.15	3.15	3.15
8	5.16	5.66	5.70	5.70	5.70	5.70	5.70	3.59	3.59	3.59	3.59
10	5.15	5.84	5.98	6.04	6.05	6.05	6.05	4.07	4.09	4.09	4.09
12	5.07	5.84	6.06	6.20	6.28	6.28	6.28	4.25	4.32	4.36	4.36
14	5.11	5.97	6.24	6.44	6.64	6.67	6.67	4.21	4.40	4.49	4.51
16	5.21	6.17	6.50	6.75	7.04	7.17	7.17	4.20	4.46	4.64	4.68
18	5.27	6.35	6.76	7.04	7.43	7.68	7.71	4.19	4.53	4.78	4.87
20	5.34	6.52	6.96	7.32	7.83	8.18	8.27	4.19	4.60	4.93	5.05
22	5.40	6.67	7.18	7.62	8.21	8.71	8.87	4.19	4.64	5.05	5.22
24	5.49	6.83	7.38	7.85	8.58	9.21	9.47	4.25	4.73	5.21	5.45

EFFECTIVE WIDTH OF SLABS WITH MAIN REINFORCEMENT PERPENDICULAR TO DIRECTION OF TRAFFIC

Concrete slabs in which the main reinforcement is perpendicular to the direction of traffic are used in through girder bridges, deck girder or T-beam bridges and in slab floors supported on longitudinal steel joists. For such designs the problem of load distribution is entirely different from that which has been outlined in the case of slabs which have their main reinforcement parallel to the direction of traffic.

The maximum bending moment in a slab, the span of which is at right angles to the direction of traffic, is determined by one of several different conditions of loading, depending upon the transverse spacing of the longitudinal beams or joists and the number of trucks for which the floor is to be designed.

In Figure 11 assumed transverse wheel spacings are shown for one and two trucks and five conditions of loading designated as Cases I to V inclusive, one of which will determine the maximum bending moment for any particular spacing of longitudinal beams; and Table 5 gives the bending moments due to unit loads for these five loading conditions.

For through girder bridges, the bending moment in the floor slab will be determined by either one or two axle loads, depending upon whether the floor is designed for one truck or two. In through girder construction the maximum range in span of the floor slabs may be taken as from 16 to 24 feet. Therefore, effective widths must be determined for one axle load (Case III) on spans from 16 to 24 feet and for two axle loads (Case V) on spans from 18 to 24 feet.

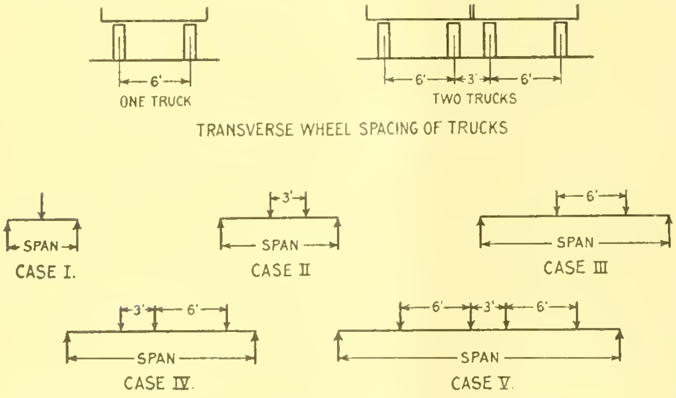


Fig. 11.—Assumed transverse wheel spacings and five conditions of loading

In a deck girder bridge designed for one truck, the bending in the floor slab will be determined by either Case I or Case III, and as shown by Table 5, Case I governs for span lengths up to 10 feet and Case III for greater span lengths. In a deck girder designed for two trucks, the bending in the slab will be determined by Case I, Case II, Case IV or Case V, depending upon the girder spacing. Case II governs for spans from 6 to 12 feet and Cases IV and V need not be considered since the girder spacing in this type of construction will seldom, if ever, exceed 12 feet.

To sum up, for both through and deck girder bridges, effective widths must be determined for various widths of wheels, as follows:

Case I, spans up to 10 feet.
Case II, spans from 6 to 12 feet.
Case III, spans from 10 to 24 feet.
Case V, spans from 18 to 24 feet.

The minimum effective width for a slab centrally loaded with a single load will occur when the slab width is a minimum and the span length a maximum; that is, when the ratio of width to span is a minimum. For the type of construction under consideration, when the spacing of longitudinal beams is such as to give maximum bending under one wheel load, an extreme assumption is that of a slab of equal span and width. For this case the value of $\frac{b}{S}$ equals 1.0 and the corresponding value of $\frac{e}{S}$, from Table I, is 0.67. Since this assumed condition is extreme and since axle spacings are so great that the preceding or following wheel loads will have no effect on the distribution, we will provide amply for all ordinary cases by assuming

$$\frac{e}{S}=k=0.70$$

Table 5.—*Maximum bending moments for various spans due to unit loads for five conditions of loading. Main reinforcemen, perpendicular to direction of traffic*

Span in feet	Bending moments in foot-pounds				
	Case I	Case II	Case III	Case IV	Case V
2	0.50				
3	.75				
4	1.00				
5	1.25	1.23			
6	1.50	1.69			
7	1.75	2.16			
8	2.00	2.64			
9	2.25	3.13	2.0		
10	2.50	3.61	2.45		
11		4.11	2.91	3.82	
12		4.59	3.38	4.56	
14			4.32	6.05	
16			5.28	7.55	
18			6.25	9.04	9.13
20			7.23		11.11
22			8.20		13.11
24			9.19		15.10

Professor Young's equation for the effective width of a centrally loaded slab in which the main reinforcement is perpendicular to the direction of traffic is $E=k(S+w)$.

Substituting the above value of k in this equation we obtain $E=0.7(S+w)$ which may be used for the determination of effective width for slabs in which the maximum bending is determined by one wheel load, as in Case I.

In Cases II, III, and V the maximum bending is determined by either two or four wheel loads and the above equation, which is for central loading, can not apply since, when the loads are placed in the position to produce maximum bending, none of them is at the center of the span of the slab. In dealing with these cases, therefore, it will be necessary to consider loads not centrally located on the span and in order to do this it will be convenient to make use of an equation proposed by Professor Young in a paper published in the Engineering News in 1914.[5] For the conditions shown in Figure 12 this equation is

$$E=2\left(x+\frac{w}{2}\right)\tan\theta+d$$

in which E = effective width

w and d = dimensions of bearing area of wheel

$\tan\theta=k$

x = distance from center of wheel to near support

When $\tan\theta=0.7$ and $d=0$ this reduces to

$$E=1.4x+0.7w \tag{8}$$

In the following discussion this equation will be used for the determination of effective width for single loads on slabs in which the main reinforcement is perpendicular to the direction of traffic.

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[5] The effective width of Reinforced Concrete Slabs Supporting Concentrated Loads. By C. R. Young. Engineering News, July 30, 1914.

Figure 13 shows the case of two equal loads, P, in the position to produce maximum bending in the slab of span, S. From the above equation the effective width for the load at point 1 is $E_1 = 1.4b + 0.7w$, and for the load at point 2 it is $E_2 = 1.4c + 0.7w$. These effective widths are the widths of slab, transverse to the direction of the slab span, over which the respective loads may be assumed to be uniformly distributed.

Therefore, at point 1 the load on a strip of the slab 1 foot in width is $\frac{P}{E_1}$ and at point 2 the load carried by a 1 foot strip is $\frac{P}{E_2}$, E_1 and E_2 being expressed in feet.

Let M_1 = bending moment on a strip of slab 1 foot in width.

Then

$$M_1 = \left[\frac{\frac{Pb}{E_1} + \frac{P(a+b)}{E_2}}{S}\right] c.$$

Let M = total bending due to the two equal wheel loads.

Then

$$M = \frac{P(a+2b)c}{S}.$$

If M is the total bending in the slab and M_1 is the bending in a strip 1 foot wide, it follows that the width in feet over which the total bending is distributed; in other words, the effective width is equal to $\frac{M}{M_1}$.

A typical determination of effective width in accordance with the above assumptions will now be given in detail.

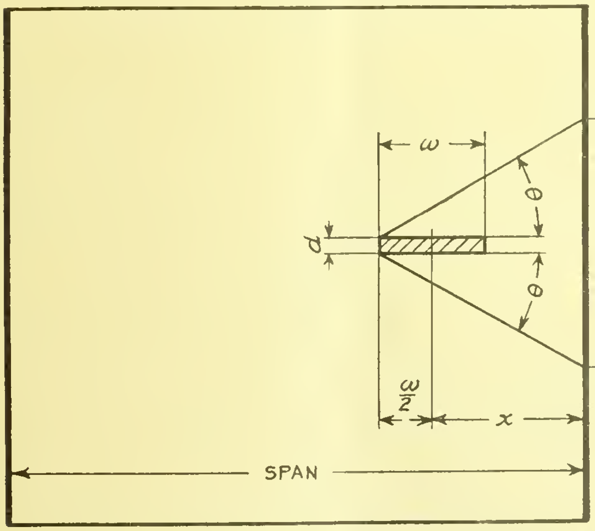


FIG. 12.—Diagram of effective width of slab with main reinforcement perpendicular to direction of traffic and load not at center of span

In Figure 13, assume

$$P = 1 \text{ and}$$

$$S = 24.0 \text{ feet}$$

$$a = 6.0 \text{ feet}$$

$$w = 1.0 \text{ foot}$$

Then

$$b = 7.5 \text{ feet}$$

and

$$c = 10.5 \text{ feet}$$

$$E_1 = (1.4 \times 7.5) + (0.7 \times 1.0) = 11.2 \text{ feet}$$

$$E_2 = (1.4 \times 10.5) + (0.7 \times 1.0) = 15.4 \text{ feet}$$

$$M = \frac{(6.0 + 15.0)10.5}{24} = 9.19$$

$$M_1 = \left[\frac{\frac{7.5}{11.2} + \frac{6.0 + 7.5}{15.4}}{24}\right] 10.5 = .676$$

$$E = \frac{M}{M_1} = \frac{9.19}{.676} = 13.60 \text{ feet}$$

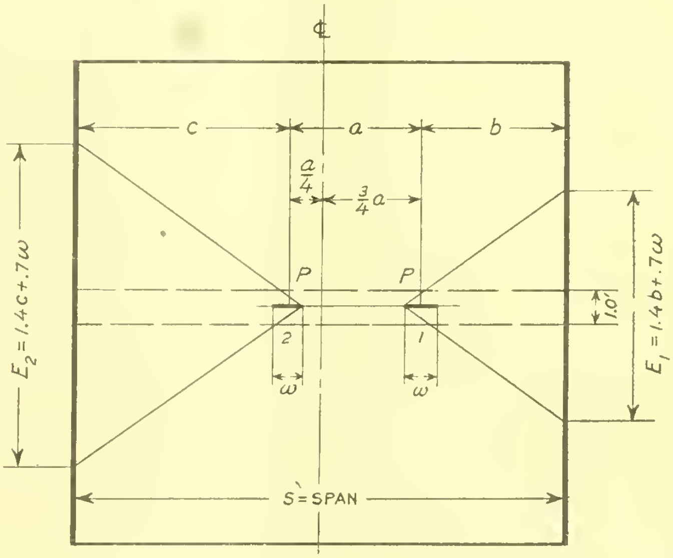


FIG. 13.—Diagram of effective widths for two loads in position to produce maximum bending on slab with main reinforcement perpendicular to direction of traffic

Following the method described above the effective widths have been computed for Cases II, III and V for various span lengths and wheel widths and the results are tabulated in Table 6. These results are also shown graphically in Figures 14 and 15, the full lines indicating calculated values of effective width and the dotted lines indicating approximate values given by straight line equations which are sufficiently accurate for purposes of design.

These approximate equations are as follows, all dimensions being in feet:

Case II, $E = 0.7(S + w) - 1.9$.
Case III, $E = 0.7(S + w) - 3.8$.
Case V, $E = 0.7(S + 1.4w) - 6.2$.

It will be noted that, for Case III, the values given by the equation are not in close agreement with the calculated values for wheels of 1 and 2 feet width on spans of less than 12 or 13 feet. The error is not important, since spans of these lengths will seldom, if ever, be encountered in practice.

SUGGESTED SPECIFICATIONS

In the foregoing discussion there have been developed equations for the determination of the distribution in concrete slabs of the bending moments due to concentrated loads. These equations are intended to provide for the range of conditions which may ordinarily be encountered in the design of highway bridges. The fundamental assumptions have been explained in sufficient detail so that, if desired, other equations to provide for other conditions may be derived.

Table 6.—*Effective width of slabs, with main reinforcement perpendicular to the direction of traffic for three conditions of loading and different tire widths*

Span (feet)	Effective width								
	Case II			Case III			Case V		
	$w=0$	$w=1.0$	$w=2.0$	$w=0$	$w=1.0$	$w=2.0$	$w=0$	$w=1.0$	$w=2.0$
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
6	2.36	3.21	3.98	------	------	------	------	------	------
8	3.70	4.45	5.18	------	------	------	------	------	------
10	5.05	5.78	6.51	3.43	4.61	5.51	------	------	------
12	6.43	7.15	7.86	4.73	5.59	6.41	------	------	------
14	------	------	------	6.05	6.84	7.61	------	------	------
16	------	------	------	7.39	8.15	8.90	------	------	------
18	------	------	------	8.75	9.49	10.23	6.40	7.72	8.8
20	------	------	------	10.12	10.84	11.57	7.79	8.81	9.74
22	------	------	------	11.48	12.20	12.92	9.16	10.07	10.92
24	------	------	------	12.87	13.60	14.30	10.56	11.39	12.23

w = width of tire in feet.

The following specifications may now be written:

Specification No. 1

In calculating bending stresses due to wheel loads on concrete slabs, no distribution in the direction of the span of the slab shall be assumed. In the direction perpendicular to the span of the slab, the wheel load shall be considered as distributed uniformly over a width of slab which is known as the "effective width."

In the following equations let—

S = span of slab.

w = width of wheel or tire.

E_1 = effective width for one wheel of a group on the same transverse element of the slab.

E_2 = total effective width for the wheel or group of wheels determining maximum bending.

All dimensions to be in feet.

Case I.—Main reinforcement parallel to direction of traffic:

(*a*) Slabs designed for one truck.
For spans of 7 feet or less, $E_1 = 0.65S + w$.
For spans of more than 7 feet, $E_1 = 0.12S + w + 3.71$.

(*b*) Slabs designed for two trucks.
For spans of 4 feet or less, $E_1 = 0.65S + w$.
For spans of more than 4 feet, $E_1 = 0.25S + w + 1.6$, with a maximum value of $(4.4 + w)$.

The sum of the effective widths for all the wheels in the group shall not exceed the width of the slab. The entire width of slab, including that portion under the curbs or railings, shall be proportioned for live load stresses as determined by the above equations. In addition to the live load and the weight of the slab itself, the edges of the slab shall be designed to provide for the additional dead load of the curbs or railings.

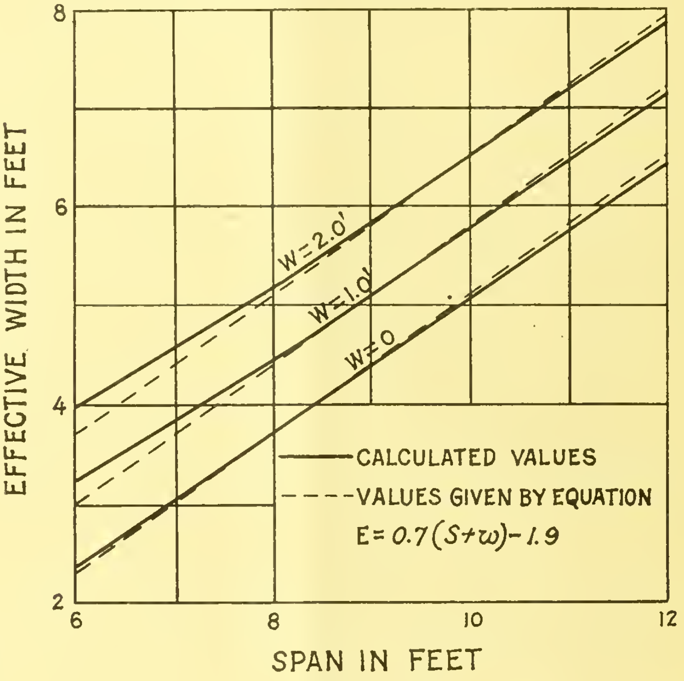
Case II.—Main reinforcement perpendicular to direction of traffic:

(*a*) Maximum bending moment determined by one wheel load, $E_2 = 0.7(S + w)$.

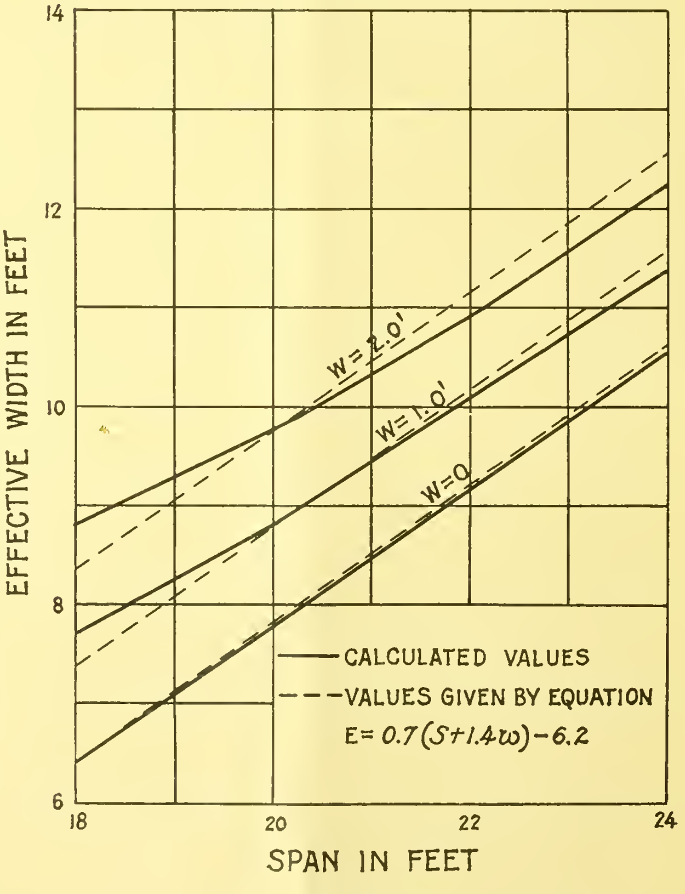
(*b*) Maximum bending moment determined by two adjacent wheels of passing trucks (wheels 3 feet c. to c.), $E_2 = 0.7(S + w) - 1.9$.

(*c*) Maximum bending moment determined by one axle load (wheels 6 feet c. to c.); $E_2 = 0.7(S + w) - 3.8$.

(*d*) Maximum bending moment determined by two axle loads (wheel spacing 6 feet, 3 feet, and 6 feet c. to c.), $E_2 = 0.7(S + 1.4w) - 6.2$.



CASE II



CASE V

Fig. 14.—Values of effective width for Cases II and V

The equations of Case II apply to slabs supported by longitudinal beams or girders and do not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges of the slab shall be supported by diaphragms or other suitable means.

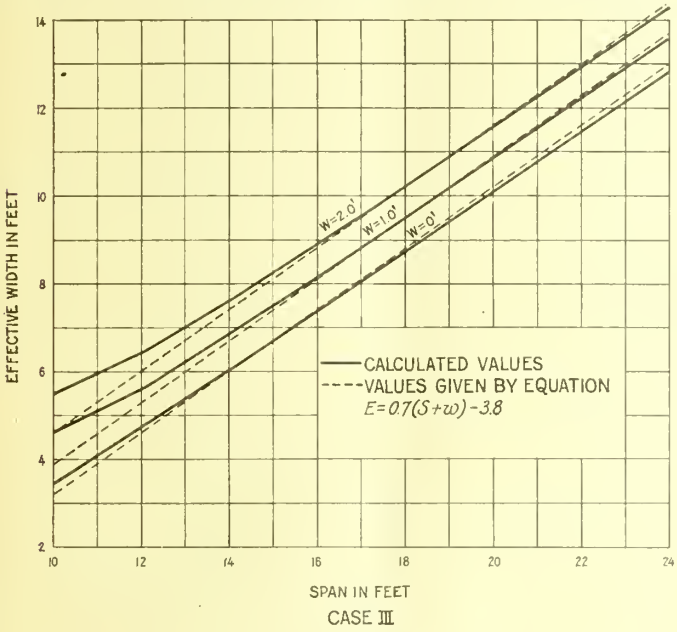


FIG. 15.—Values of effective width for Case III

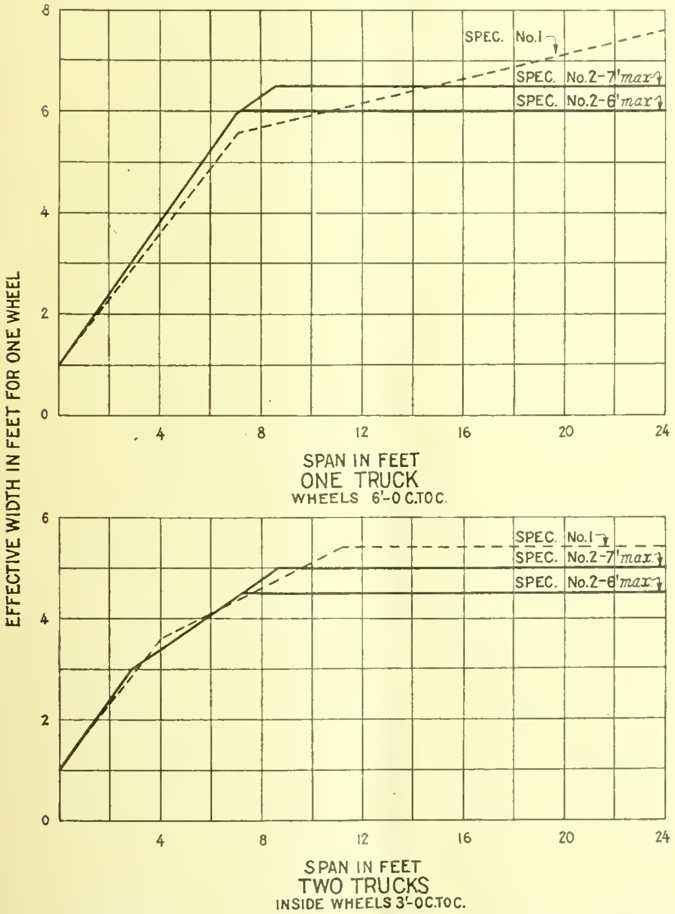


FIG. 16.—Comparison of specifications. Main reinforcement parallel to direction of traffic. Wheel width, 12 inches

The foregoing requirements are somewhat involved and cumbersome for the purpose of a design specification and, furthermore, are restricted in application to the particular conditions of loading which have been considered. The following specification is less cumbersome, more general in application, and gives results which are considered as being in sufficiently close agreement with those of Specification No. 1.

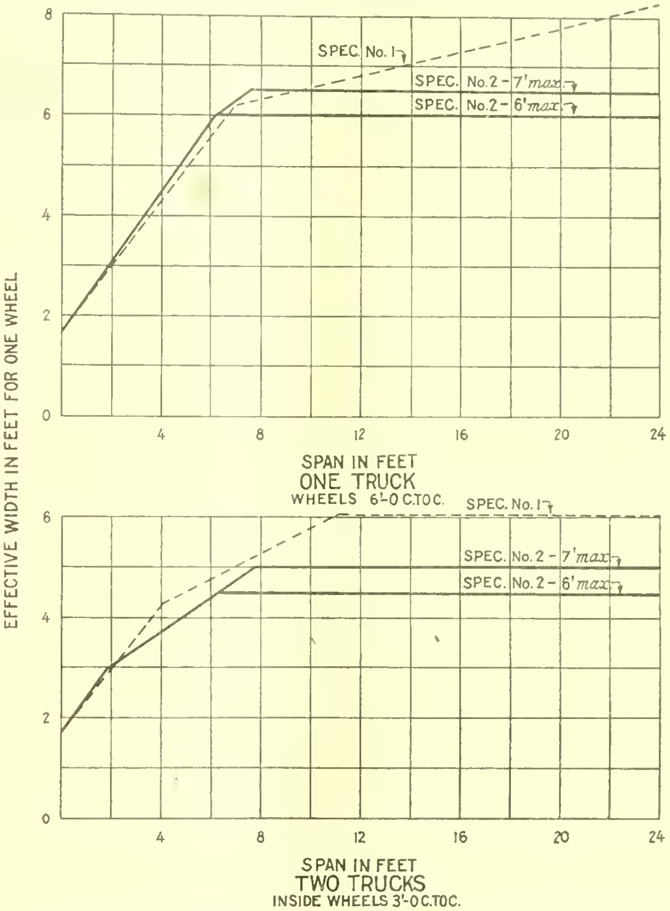


FIG. 17.—Comparison of specifications. Main reinforcement parallel to direction of traffic. Wheel widths, 20 inches

SPECIFICATION No. 2

In calculating bending stresses due to wheel loads on concrete slabs, no distribution in the direction of the span of the slab shall be assumed. In the direction perpendicular to the span of the slab the wheel load shall be considered as distributed uniformly over a width of slab which is known as the "effective width."

In the following equations let—

S = span of slab in feet.

w = width of wheel or tire in feet.

x = distance in feet from the center of the near support to the center of wheel.

E = effective width in feet for one wheel.

Case I.—Main reinforcement parallel to direction of traffic $E=0.7S+w$.

For this case the value of E shall not exceed 7 feet.

(Continued on page 24)

EFFICIENCY IN CONCRETE ROAD CONSTRUCTION

A Report of Observations Made by the Division of Control, Bureau of Public Roads

Reported by J. L. HARRISON, Highway Engineer

PART V.—SPEEDING UP CONSTRUCTION WORK

The preceding articles in this series have dealt with losses of time while work is in progress. In addition to these losses, it has been found that on most jobs a surprising amount of time is lost through failure to work at all. Work is often begun a little late in the morning. Often it is stopped too early at night. Unpleasant weather is permitted to interrupt. Wet weather unduly delays operations; in many cases because there has been a failure to keep the roads in a condition which minimizes the effect of heavy rainfall. These and the lesser causes which have been found to influence the length of the working-day accumulate a total of time lost which becomes a material factor in determining the amount of work which can be done during a normal season.

The bureau's study of these losses has not been completed; therefore no final conclusions will be drawn. However, as illustrating how important they may be, two cases from the Middle West will be cited. In each case the records given below are for the month of July last; and, as on each job one Sunday was used for work, the maximum working time possible was 27 days.

Job A.—Working-day, 10 hours:	
Time worked, hours	225
Time lost because of rain, hours	18
Other time lost, hours	27
Job B.—Working-day, 11 hours:	
Time worked, hours	155
Time lost because of rain, hours	22
Other lost time, hours	120

To what extent the "other time losses"—in one case 27 hours and in the other 120 hours—are due to managerial laxity can not be definitely stated at this time but the data already in hand warrant the conclusion that a good deal of it can be properly placed under no other heading. It may, perhaps, be added that the general data bearing on this matter suggest that time lost because of managerial laxity, that is, time allowed to pass without even so much as an effort to produce, appears to amount to approximately two hours a day. Averages of this sort are of little value to the contractor who must, of course, deal with conditions as they are on his particular job rather than as they are found to prevail generally. The general figure is referred to here only because it points to a really serious condition that is affecting the production of paving plants more than most contractors suppose.

The facts given above and in the preceding articles deal with the losses of time which the bureau's studies in this field have developed. But, in the last analysis, it is of little value to know what losses of time are occurring on a construction job, unless this knowledge can be utilized for their elimination and through their elimination for the improvement of production. Moreover, until such use has been made of the data, there is ground for reasonable doubt as to whether an effort to so modify common practices as to eliminate or reduce these time losses will not at once develop new difficulties, as great as or greater than those which it is sought to correct. Thus, the writer has often been cautioned that the mixer time can not be saved by fully overlapping the discharge and the raising of the skip because the power plant is not adequate to start the skip and the discharge mechanism together while driving a full drum. Of course, there is no such inadequacy of power on any good mixer but more than once both the operator and the contractor have shown honest fear as to the safety of the mixer until the demonstration has progressed for some time. On other occasions a fear has been expressed that the puddlers or the finishers or other elements in the organization can not keep up if production is increased.

Such doubts are natural enough, and while the assurance that their lack of validity can be developed when the conclusions based on the time studies are put to a test increasingly dominates the viewpoint of the investigator as the mass of data at his disposal increases, it must nevertheless be true for him as for the contractor that his case is not fully established until the accuracy of his conclusions has been subjected to the test of application under ordinary working conditions. Indeed, it was this view of the matter that brought about the decision to attempt a field application of these studies. The data which had been collected showed a general condition of low efficiency in production. The causes were, in a general way, common to all projects and their rectification had every appearance of being simple enough. But, no matter what appearances may be, a decision that current conditions are properly to be laid to inefficiency can not reasonably be made until it has been shown that the unsatisfactory conditions which have been found to exist can readily be improved, eliminated or changed for the better.

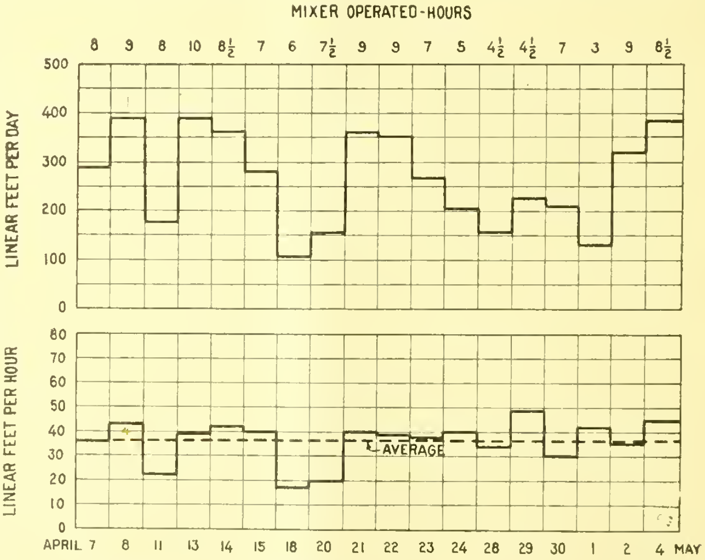


FIG. 1.—Daily and hourly production on project No. 1, under the supervision of the contractor

EFFICIENCY INCREASED ON SEVEN TEST PROJECTS

During the past year, work specifically looking toward improving the production of concrete paving plants has been done on seven projects or sections of projects in three States, under the immediate charge of four different engineers. About two months has been given to each project studied, except No. 7, as listed in the following brief summary of the results obtained:

Project	Engineer	Result
1	A	Output about doubled. Production limited by an inadequate truck supply. For greater detail see Figures 1, 2, 3, and 4, and Tables 1 and 2.
2	B	Output increased materially. Maximum run under contractor's management 683 feet. Maximum run under suggestions based on studies 944 feet. Work on this section was discontinued because truck supply was inadequate to serve both project 2 and project 3 (two sections of the same contract) and it was desired to have an adequate transportation supply on project 3.
3	A	Output about doubled. Maximum production 101 feet per hour for 12½ hours. Average production during last week of study 97 feet per hour for 61 hours. For greater detail see Figures 5, 6, and 7 and Tables 3 and 4.
4	C	Efficiency of truck operation increased from about 73 to about 95 per cent. Production correspondingly increased. Truck supply inadequate during whole of study. For additional data see Figure 8.
5 6	A D	Two sections of the same contract. Production on both sections was consistently increased in spite of inadequate transportation, which the contractor did not see fit to remedy, and exceptionally adverse weather conditions which prevented the development of really first-class working organizations and caused a direct loss of 69½ hours of working time during the month of September. Notwithstanding these difficulties, the output on the first section (5) was increased from 9,269 feet in 170¼ hours of mixer operation during the month of August to 12,796 feet in 160 hours of mixer operation during the month of September, an increase from 57 to 80 feet per hour. On the second section the maximum production was increased from 406 feet in 9 hours of mixer operation to 1,127 feet in 12½ hours of mixer operation.
7	A	Contractor's low production found to be almost entirely due to functional inadequacy of mixer and contractor so informed. No effort was made to speed up this project.

Two plans of operation have been tried on this work: (1) Assistance to the contractor, rendered through suggestions as to appropriate changes in methods and practices; and (2) placing at the contractor's disposal an engineer competent to act as superintendent who thereafter, during the test period, took over a considerable part of the work of the superintendent in directing operations on the project. It is hardly necessary to point out that there are vital differences between these plans. The first depends on the salesmanship of the engineer. His data may be perfectly correct and the conclusions which he draws from the data may be unassailable, but unless he can convince the contractor and his superintendent, little or nothing is done.

It has been the uniform experience of the bureau's representatives that contractors assume that the results which they are securing are the best that anyone can secure under existing conditions, and that uncontrollable adverse physical conditions or others differing from the conditions prevailing on some other job are responsible for the low production record. It may be admitted that in occasional instances this is correct. Jobs 5 and 6 offer an illustration somewhat in point. These jobs were undertaken toward the end of the construction season and weather conditions became so adverse that, though improvement in output resulted, it was impossible to maintain a smooth-running organization, and so fully to develop this improvement. But even in this case, it would be improper to assume that the problem did not permit of solution by older or more experienced men. On the other hand, in the great majority of cases, it is apparent to the impartial and disinterested observer, that physical conditions are a small factor in determining the relative efficiency of production on different projects. Commonly the quality of the superintendence is the factor of first importance. The adequacy of the plant also is a factor but it would appear to be a fair presumption that if the superintendence is really first-class, inadequacies in plant and equipment will be corrected within a reasonable period.

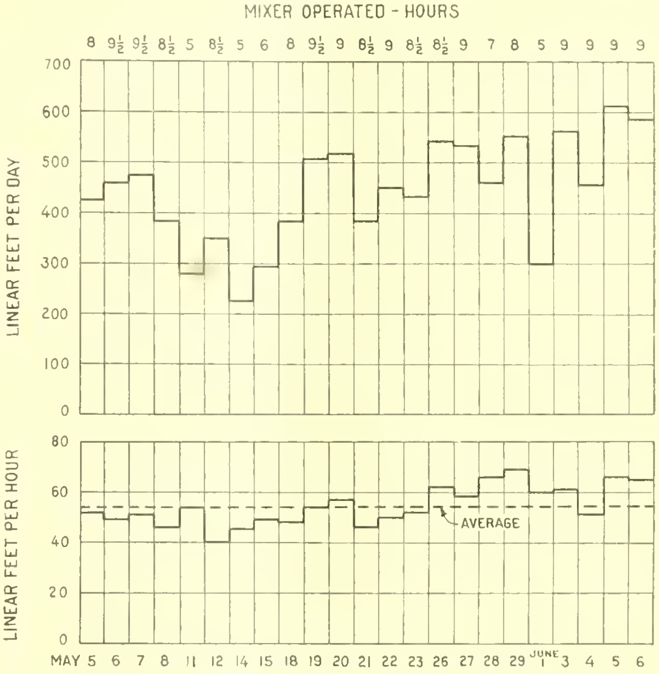


FIG. 2.—Daily and hourly production on project No. 1 under suggestions made by the bureau engineer

But as the engineer, endeavoring to improve production by offering suggestions, must convince a man who is disposed to lay his troubles to almost anything other than their real cause, the problem of initiating improved methods on the basis of pure salesmanship will be understood to be a difficult one. Results have been secured on this basis but it is by no means as uniformly effective as placing on the job a man who, with the full consent of the contractor, will take over the direction of personnel and equipment so far as this is necessary in order to place the modified practices in operation.

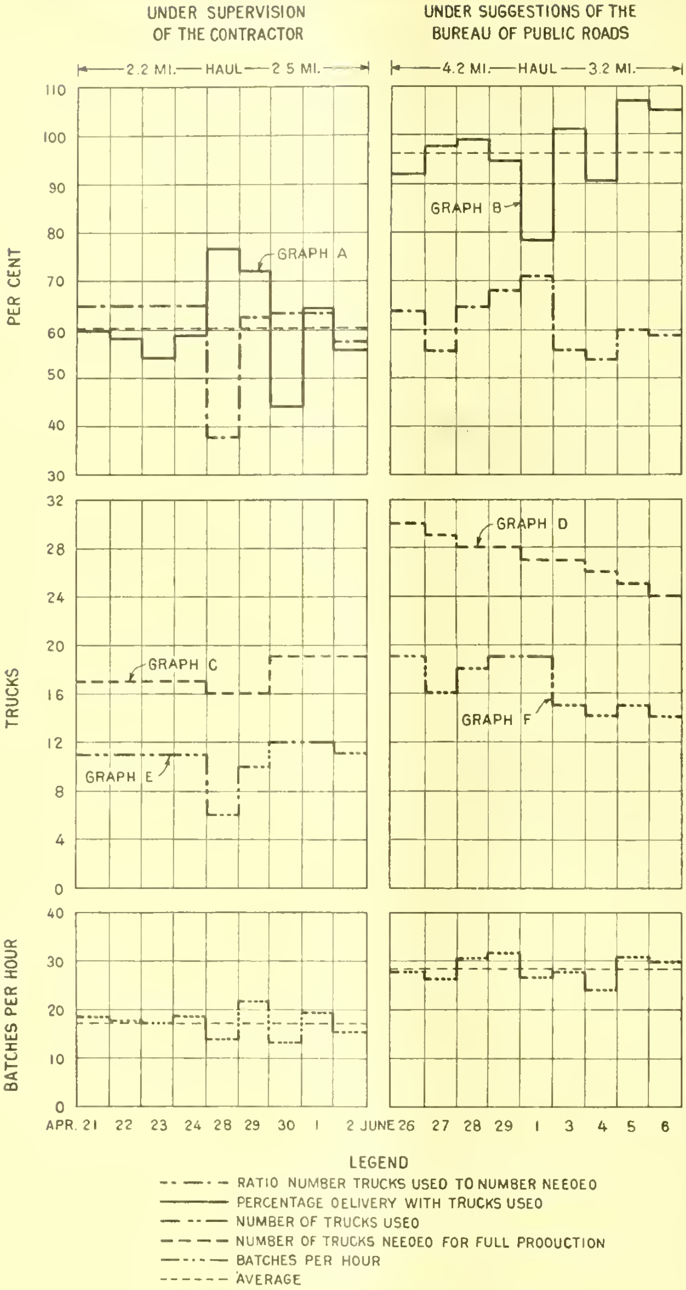


Fig. 3.—Study of truck operation on project No. 1

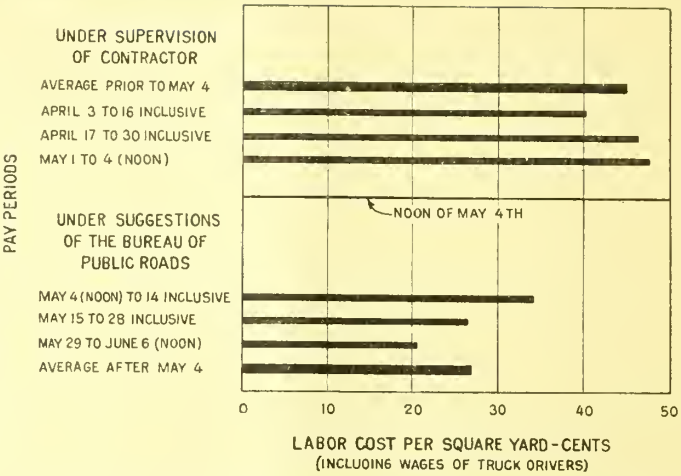


Fig. 4.—Effect of increase in production on labor cost (including wages of truck drivers) on project No. 1. The increase in production resulted from the suggestions of bureau engineer

When the latter method is used the new methods sell themselves as their application increasingly develops production and it has commonly been found that as production has increased, confidence in the merit of the new practices has encouraged the contractor to facilitate their perfection in every possible way.

GRADUAL SIMULTANEOUS IMPROVEMENT OF ALL OPERATIONS THE AIM OF TESTS

In the application of data obtained from time studies, the practice has been to make a gradual correction in operation affecting all related parts of the improvement program simultaneously. This has been found to be desirable, if not necessary. The major difficulties on most concrete jobs are at the mixer and in the material supply. To correct mixer operation without modifying the material supply ordinarily will have no effect on production. On the other hand, unless there is a shortage in the material supply, to improve practices in handling the trucks without improving the operation of the mixer will not improve production. But the objective is improved production—not a theoretical perfection in operating practices—so both operations must be improved together or the program tends to discredit itself for the simple reason that it becomes obvious to all that the objective is not being reached.

One other comment in regard to this work may be appropriate, namely, that when an effort is made to improve production the extent to which this can be accomplished will depend on the equipment at hand. There is a difference here between the work of the engineer whose problem it is to demonstrate efficient methods with the equipment available and that of the superintendent, one of whose normal functions it is to determine what equipment is required and to procure and maintain a full supply of such equipment.

The engineer who is endeavoring to increase production may, of course, make every effort to induce the contractor to obtain such equipment as may be needed in order to obtain full production. But often conditions are such that this equipment can not be obtained at once. The criterion of his success must, therefore, be the amount of correction that can be obtained in operating efficiency using the equipment with which prevailing conditions force him to work. As a result, it is not always possible to secure a full correction of the mixer output even though improvement in the operating efficiency of the available truck supply is quite satisfactory. The graphs in Figure 8, which are for project No. 4, bring out this point. Graph A shows the efficiency of truck operation under the contractor's management, which had been at least as good as the average. Graph B shows the efficiency of truck operation as effected by suggestions made by the bureau's representative. In this case, the truck supply was inadequate for the whole period during which suggestions were being made; but the production per hour of work increased, partly because of the shortening haul which naturally brought mixer output capacity and truck delivery capacity closer together and partly because of the greater efficiency of truck operation. Toward the end of the period production per hour rose high enough to force the mixer, and the improvement is clearly apparent from a study of graphs C and D of Figure 8, which show the number of batches placed per hour before the suggestions were made and toward the end of the bureau's connection with the job.

Another job, project No. 1, of similar characteristics as to truck supply but with a lower original efficiency in operation, is represented by the graphs of Figure 3. Initially the haul was about 1.5 miles. This increased during the next few days to a maximum of about 3.2 miles which completed that section. The mixer was then moved to a point to which the haul was 4.9 miles, from which it was reduced as the work progressed to 3.2 miles at the completion of paving. This job differed from project No. 4, referred to above, in that the bureau's engineer was able to persuade the contractor to rent a few extra trucks though not enough to enable him to make a delivery equal to the mixer's output capacity. However, the contractor let these trucks go too soon. Under the contractor's management operating efficiency at the mixer was low. This was complicated by an unfair setting of the mixing time as evidenced by the high mixing time shown in Table 1. In addition, however, the general efficiency prevailing on all parts of the job was considerably below normal. Increasing the efficiency with which the trucks operate (compare graphs A and B, fig. 3) improved the supply of material at the mixer, and cutting out the time losses at the mixer increased the mixer's capacity to handle the material delivered to it. As a result production rose steadily (time losses due primarily to weather conditions being allowed for). As in the first case cited, the material supply during the latter part of the work was also influenced by the decreasing haul, but as truck supply was never sufficient (compare graphs C and D with E and F in fig 3) the output of the mixer never reached its full capacity. The increased output was secured without material change in personnel employed. Figure 4 shows the effect of this increased production on the unit labor cost (including truck drivers) of this work

RECOMMENDATIONS OF THE BUREAU CONVINCINGLY DEMONSTRATED

On project No. 3, as listed above, transportation facilities were adequate and working conditions, weather, etc., normal. The contractor had maintained a rate of production which would be considered in most places, at least, as quite satisfactory. (See fig. 5.) In this respect this job differed widely from project No. 1, just noted. On that job it has been possible to raise production from subnormal to about normal, but because of the limited truck supply available it was impossible to get really first-class production from the mixer, though this job, as well as project No. 4, showed that the efficiency with which the transportation was operated could be maintained at an

TABLE 1.—*Results of stop-watch studies on job No. 1 under the contractor's management before suggestions were made by the bureau*

Study No.	Number of batches per hour	Mixer time				Lost time								
		Charge	Mix	Discharge	Percentage of total working time	Truck shortage	Slow truck operation	Mixer trouble	Slow mixer operation	Water supply	Setting dowel rods	Preparing sub-grade	Lack of cement at mixer	Dumping cement into skip
		Seconds	*Seconds*	*Seconds*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
49	22	11	78	23	68		3		27					2
50	22	11	78	21	68			3	22				5	2
51	13	10½	79	21½	41		46		9					4
52	12	10½	76½	21	36		39	9	10	2				4
53	9	11	76½	21	27	15	44		8					6
54	20	11	76	21	60	13	10		14					3
55	16	10½	76	22	49				23		6	21		1
56	17	10½	71½	20	48				21			31		
57	14	10½	79½	21	43		41		10					6
58	15	10	78	20	45		45		2					8
59	20	10½	80½	20	62		19		14					5
60	20	10½	79½	20½	61				23	7			5	4
61	19	11	77	20	58		2	15	7		5	7	6	
Average	16.9	10.7	77.4	20.9										
Percentage					51.2	2.2	19.2	2.1	14.6	0.7	0.8	4.5	1.2	3.5

Total time per batch $=\frac{3,600}{16.9}=213$ seconds. Time per batch for maximum efficiency (48 batches per hour) = 75 seconds. Actual operating efficiency $=\frac{75}{213}=35.2$ per cent.

TABLE 2.—*Results of stop-watch studies on job No. 1 operated in accordance with suggestions made by the bureau*

Study No.	Number of batches per hour	Mixer time				Lost time			
		Charge	Mix	Discharge	Percentage of total working time	Truck Shortage	Mechanical truck trouble	Slow mixer operation	Water supply
		Seconds	*Seconds*	*Seconds*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
60	28	10	72	8	70		17	3	10
61	26	10	71	8	64	11	16	3	6
62	25	10	68½	8	60	37	3		
63	31	10	68	8	75	23	1	1	
64	23	10	71	8	57	20	20	3	
65	31	10	51	8	60	37	1	2	
66	30	10	51	8	57	34	7	2	
67	28	10	59	8	60	35	4	1	
68	30	10	64	8	68	25	1		6
Average	28	10	63.9	8					
Percentage					63.3	24.7	7.8	1.8	2.4

Total time per batch $=\frac{3,600}{28}=129$ seconds. Time per batch for maximum efficiency (48 batches per hour) = 75 seconds. Actual operating efficiency $=\frac{75}{129}=58.2$ per cent

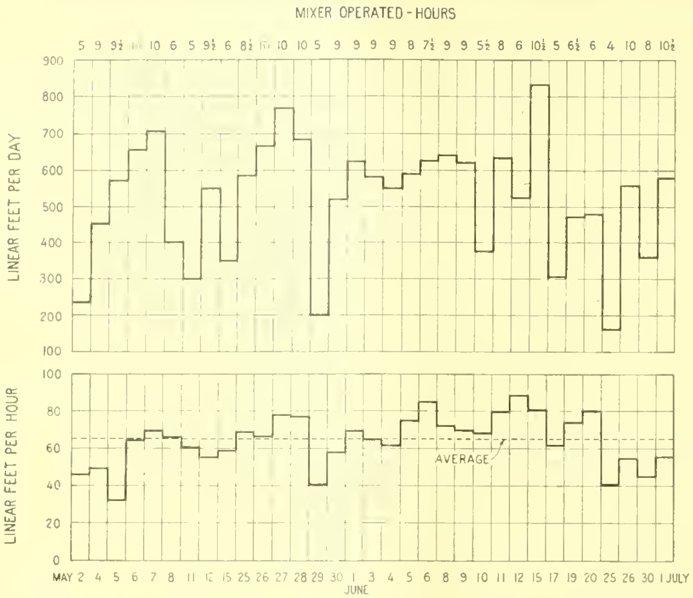


Fig. 5.—Daily and hourly production on project No. 3, under the supervision of the contractor

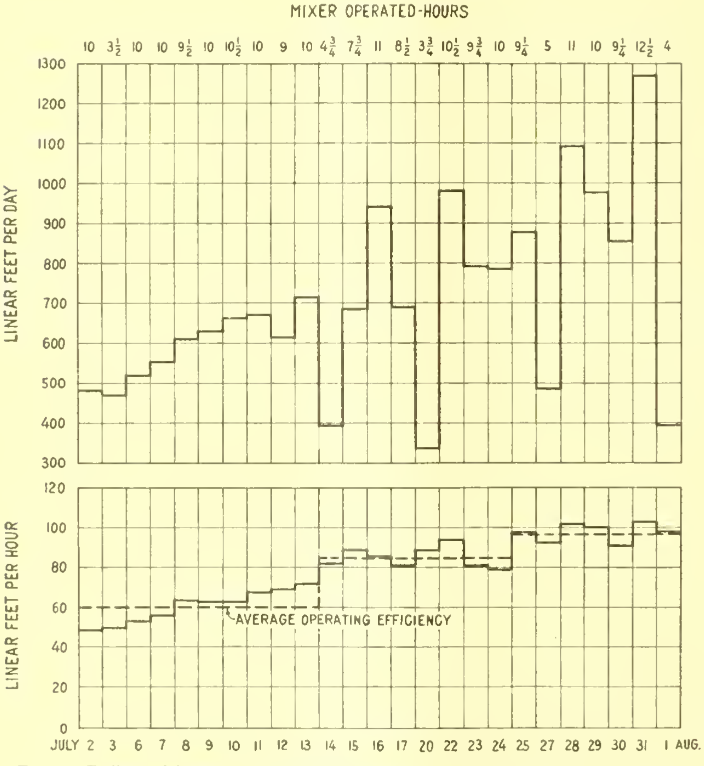


Fig. 6.—Daily and hourly production on project No. 3 under suggestions made by the bureau engineer

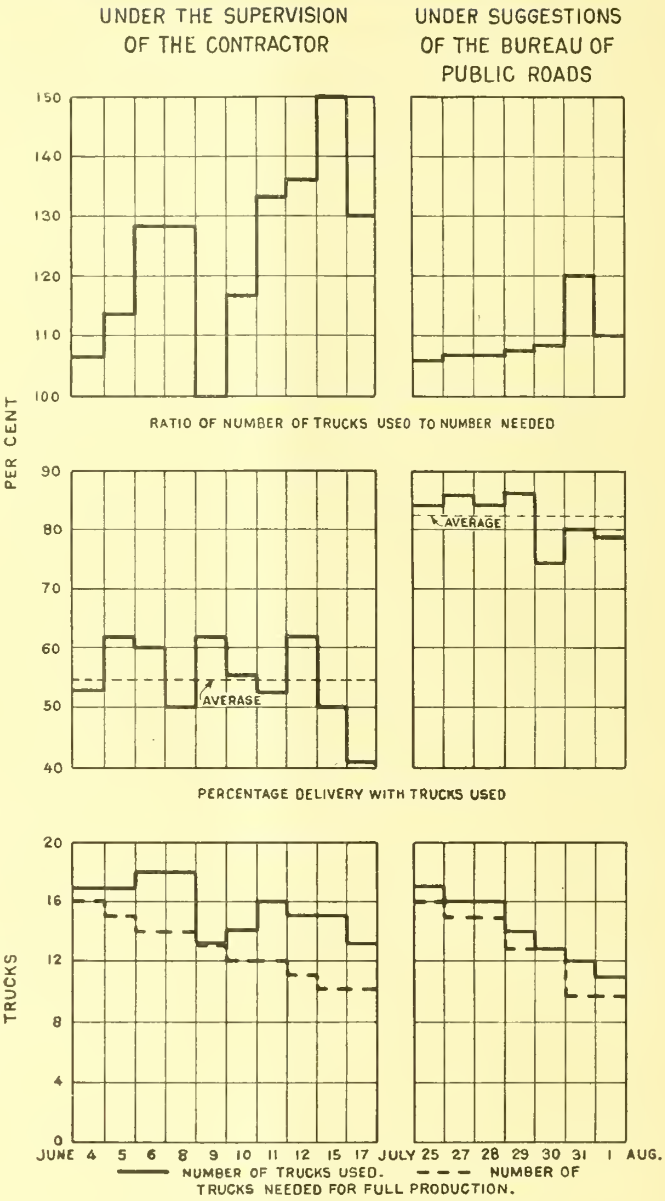


Fig. 7.—Study of truck operation on project No. 3

average above 90 per cent of theoretically perfect performance. As it was evident that project No. 3 could provide an adequate transportation supply it was apparent that it would be possible to use it to ascertain: (1) Whether efficiency at the mixer could be developed to, and consistently maintained at, over 90 per cent of theoretical efficiency; (2) whether the efficiency of material delivery could be maintained at a correspondingly high level while such a high rate of demand was being made on it; (3) whether in the face of such a rate of output any other unit of the equipment would prove inadequate; or (4) whether any part of the labor organization would have to be enlarged.

Operations were begun simultaneously at the mixer where the operator was carefully trained to avoid time losses, at the material yards where cement handling had been causing some delay in operating the trucks, and on the truck supply where miscellaneous losses had amounted to about six minutes per trip. On this particular job production developed rather slowly (fig. 6), the reason being that it took a good deal of time to train the truck drivers to operate on a schedule which would deliver enough material to force the mixer. At first there was some actual truck shortage, but this was soon corrected, and as the delivery increased the mixer operator made increasing efforts to master the problem of handling it.

The demonstration was a complete success. Figure 6 shows the increase in production; and Figure 7 shows the characteristic differences between the contractor's operation of trucks and the operation developed by the bureau's representative for the same length of haul. Tables 3 and 4 show the characteristics of operation under the contractor's management, and, during the last week of the test period, under the suggestions of the bureau. The maximum rate of production was attained on the last full day's operation when 101 feet per hour were laid for 12½ hours. During this period, which was long enough to test the effectiveness of operating practices, no time was lost on account of functional inadequacy in any unit of the mechanical equipment, thus demonstrating conclusively that the equipment correlated with the mixer had ample capacity. The truck supply was adequate and its performance demonstrated that trucks are capable of showing and of maintaining a high rate of efficiency if their operation is properly supervised.

ADVANTAGE OF HIGH-CLASS SUPERINTENDENCE EMPHASIZED

Possibly an even better demonstration of the real capacity of a paving outfit is offered by the performance during the last week (61 working hours) of the bureau's contact with this project. During this period production amounted to 5,925 feet, which is an average of 97.1 feet per hour worked. The lowest rate of production during this period was 92.4 feet per hour and the highest 101 feet per hour, an operation of notable uniformity at a high rate of efficiency which fully demonstrates that the well balanced paving outfit is capable of sustained high production if the various elements in the construction organization are correlated and synchronized to eliminate time losses.

The experience on this job points directly to the last conclusion it is desired to draw at this time, namely, that job management, or superintendence, is commonly the weakest point in the construction organization. Contractors are losing thousands of dollars every year because they employ as superintendents men who have proved to be good foremen or who have been found to be bright inspectors on their jobs. The point that is commonly overlooked is that the efficient field supervision of a large construction job requires a breadth of knowledge and an ability to analyze, arrange, and synchronize the elements of a construction opera-

TABLE 3.—*Results of stop-watch studies on job No. 3 under the contractor's management before suggestions were made by the bureau*

Date	Number of batches per hour	Mixer time				Lost time							
		Charge	Mix	Discharge	Percentage of total working time	Truck shortage	Mechanical truck trouble	Mixer trouble	Slow mixer operation	Water supply	Preparing subgrade	Delay due to concrete finishers	Miscellaneous delays
1925		*Seconds*	*Seconds*	*Seconds*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
June 11	16	9½	59½	10	35				15			50	
12	36	9½	61	10	80½				10½		9		
12	31	9	60	12	69½		9		16½		5		
15	36	9½	61	10	81		9½		6	2	1½		
15	34	9½	61½	10	76		5		13	6			
19	31	9	58	12	67		3		17½		5½	7	
20	38	10	59	10	82	10	1		6	1			
20	38	10	59	10	82½	6			8	2	1½		
20	31	9½	59	10	67	14	1		9				9
25	10	10	61	10	23	13	21½	41½	1				
26	17	9½	61	10	38	28	1	27½	1½	4			
26	24	9	60	12	54	39½			4½	2			
30	20	9½	61	10	44	49	3		3	1			
30	23	9	61	12	51½	30½	2		6	10			
Average	27.4	9.5	60.1	10.6									
Percentage					60.9	13.6	4.0	4.9	8.4	2.0	1.6	4.0	0.6

Total time per batch=$\frac{3,600}{27.4}$=131 seconds. Time per batch for maximum efficiency (48 batches per hour)=75 seconds. Actual operating efficiency=$\frac{75}{131}$=57.1 per cent.

TABLE 4.—*Results of stop-watch studies on job No. 3 during the last week of the bureau's assistance to the contractor*

Date	Number of batches per hour	Mixer time				Lost time					
		Charge	Mix	Discharge	Percentage of total working time	Mechanical truck trouble	Mixer trouble	Water supply	Preparing subgrade	Lack of materials due to poor supervision	Miscellaneous delays
1925		*Seconds*	*Seconds*	*Seconds*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
July 27	45	10	64	3	95½	2½	1	1			
28	44	10	64	2½	94½	5		½			
28	43	10	63½	2½	91	8½					½
28	44	10	64	2½	93	½				6½	
29	45	10	63½	3	95½			4½			
29	46	10	64	2½	99½	½					
30	38	10	64	3	81			2	4	13	
30	38	10	64	3	81	3		[1] 16			
31	47	10	64	2½	100						
31	43	10	63½	3	91½	2		1	5½		
31	46	10	63½	2½	98			2			
Aug. 1	45	10	64	2½	95½			4½			
Average	43.7	10	64	2.7							
Percentage					97.0	2.0		2.6	0.8	1.6	

[1] Because of this time loss due to high pressure in the line, the water losses do not show the improvement that was actually made.

Total time per batch=$\frac{3,600}{43.7}$=82.4 seconds. Time per batch for maximum efficiency (48 batches per hour)=75 seconds. Actual operating efficiency=$\frac{75}{82.4}$=91.0 per cent.

tion which few men possess. So much is at stake in a large construction operation, and the difference between ordinary operation and really efficient operation is so great, that really first-class superintendents, i. e., men who can obtain a general efficiency of operation exceeding 90 per cent of theoretical perfection, are worth, to a large contractor, almost anything they care to ask, while men of ordinary ability are expensive at any price. Indeed it is no exaggeration to say that the first-class superintendent can secure at least twice the production which is now commonly obtained with little or no increase in labor or in equipment charges, which is equivalent to saying that he can reduce by half the ordinary unit labor and equipment charges that enter into the cost of a concrete pavement. Contractors would be well advised to give more thought to this phase of their work for, under conditions generating keen competition, weakness at this point in the construction organization is an invitation to financial ruin.

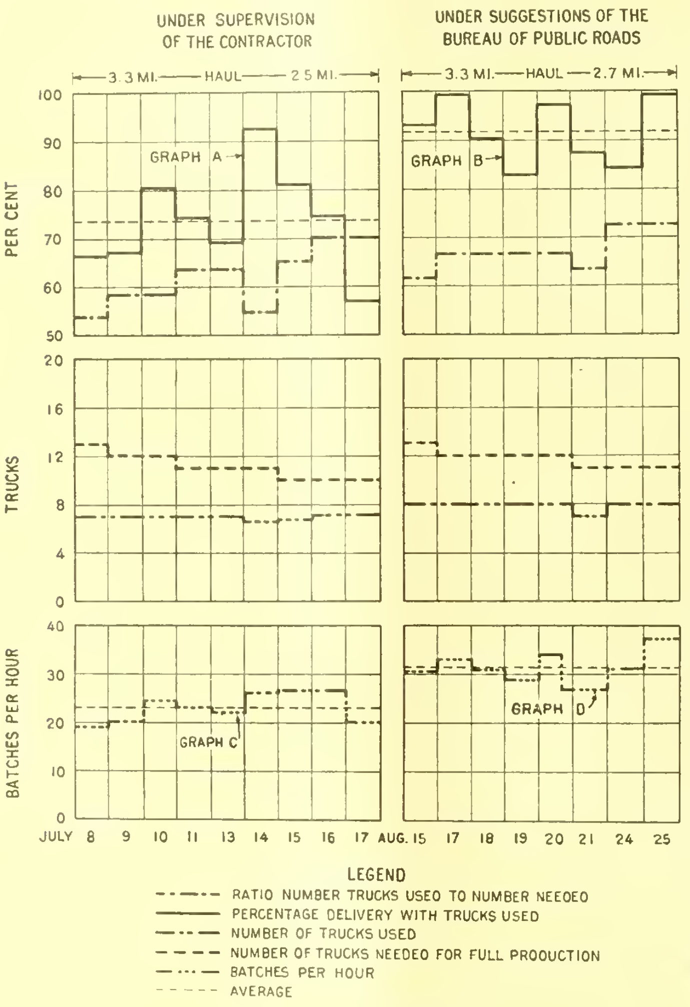


Fig. 8.—Study of truck operation on project No. 4

(Continued from page 17)

When two wheels are so located on a transverse element of the slab that their effective widths overlap, the effective width for each wheel shall be $\frac{1}{2}(E+a)$, where a is the distance between centers of wheels.

Case II.—Main reinforcement perpendicular to direction of traffic, $E=0.7(2x+w)$.

For this case the bending moment on a strip of slab 1 foot in width shall be determined by placing the wheel loads in the position to produce maximum bending; determining the effective width for each wheel; and assuming the load delivered by each wheel to the 1-foot strip to be the wheel load divided by its respective effective width.

This design assumption does not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges of the slab shall be supported by diaphragms or other suitable means.

It will be noted that specification No. 2 is somewhat similar to certain specifications now in general use, particularly as regards the requirements for Case I. However, the usual specification of this type limits the effective width to a maximum of 6 feet and it is felt that this requirement is more severe than is necessary. A maximum of 7 feet gives results which, for ordinary widths of wheel, are in closer agreement with those given by specification No. 1. Wheel widths of from 12 to 20 inches practically cover the range of widths which are usually assumed for design purposes. In Figures 16 and 17 comparisons are shown for Case I, between specification No. 1 and specification No. 2 for wheel widths of 12 and 20 inches and, in the case of specification No. 2, for both a 6-foot and a 7-foot maximum. For Case II the comparison of the two specifications is shown in Figures 14 and 15, the dotted lines representing specification No. 1 and the full lines specification No. 2.

It is realized that certain of the basic assumptions which have been made for the derivation of these specification requirements are open to serious criticism and that future research may discredit the conclusions which have been reached. One of the assumptions which might be questioned is Professor Young's basic equation which has been reduced to the form, $E=0.7(2x+w)$. The use of this equation results in a seeming discrepancy in that the live load bending moment on a 1-foot strip of slab loaded at the center is less than that due to the same load placed at the quarter point. In spite of this discrepancy the equation has been accepted for lack of data upon which to base one which might, perhaps, lead to more logical results.

The foregoing study is an attempt to apply the available test data in as logical a manner as possible to the variety of conditions encountered in the design of the floor slabs of highway bridges and the specifications are suggested for use only until such time as further research may increase our knowledge of the subject

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ANNUAL REPORT

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

FEBRUARY 28, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 9,448,345.63	$ 4,513,891.67	522.6	$ 6,638,162.30	$ 3,164,382.03	318.3	$ 760,674.71	$ 335,612.73	16.6	$ 3,472,470.81	Alabama
Arizona	9,680,133.43	6,016,119.94	613.8	1,041,126.35	638,703.27	82.2	1,248,182.17	827,613.98	89.0	178,977.10	108,150.70	26.7	3,028,661.11	Arizona
Arkansas	13,310,190.08	6,380,181.73	1,048.9	3,827,872.61	1,684,144.69	200.9	5,760,682.93	2,766,042.82	367.7	545,641.81	240,645.74	37.4	1,534,789.02	Arkansas
California	22,346,175.99	10,719,249.51	894.2	2,893,278.84	1,327,019.89	83.8	11,688,518.51	5,777,134.17	345.9	298,080.20	107,351.38	6.0	4,148,059.95	California
Colorado	11,876,703.94	6,067,814.34	661.2	1,029,861.87	530,202.26	41.7	4,588,109.90	2,317,820.29	240.8	246,862.45	138,452.34	19.5	3,271,522.77	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	750,966.91	260,212.14	13.4	1,901,818.16	590,269.68	29.1	238,929.61	62,777.95	3.8	1,611,062.57	Connecticut
Delaware	4,281,559.21	1,496,190.65	107.1	455,744.13	212,544.95	12.3	980,042.68	388,784.75	22.7	445,559.65	9,000.00	10.5	387,537.65	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				10,346,486.92	4,918,554.60	295.5	130,171.16	65,085.58	13.7	1,696,825.85	Florida
Georgia	20,166,002.37	9,406,366.46	1,478.3	3,423,882.39	1,664,584.99	217.6	11,417,017.80	5,396,853.76	699.4	314,279.02	157,139.48	33.1	1,807,008.31	Georgia
Idaho	9,394,676.80	4,816,332.26	600.1	1,013,545.40	646,811.83	73.2	2,442,858.61	1,491,685.71	157.3	461,293.42	249,557.38	17.0	1,357,239.84	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	1,939,886.69	932,621.30	64.0	7,398,089.70	3,590,111.46	253.4				6,669,388.96	Illinois
Indiana	13,639,172.56	6,662,456.68	422.1	1,196,563.59	573,819.54	47.1	18,057,715.07	8,476,679.76	490.9	421,901.51	208,509.47	12.7	2,382,990.55	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	722,462.52	320,385.06	36.5	8,002,159.56	3,608,569.59	491.1	1,516,282.01	738,026.75	164.3	3,711,108.61	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	4,634,468.55	2,066,681.33	264.9	10,122,229.26	4,058,981.07	507.1	3,083,682.14	839,663.69	182.9	2,743,811.59	Kansas
Kentucky	14,832,324.28	6,206,994.59	584.9	3,359,707.53	1,300,700.93	92.3	7,350,016.07	3,357,893.11	312.9	193,076.46	79,641.23	2.0	2,268,579.14	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,079,810.60	503,534.86	68.9	3,901,298.42	1,900,233.11	186.0				1,588,769.17	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	516,374.59	257,101.90	19.3	1,864,665.87	734,349.40	60.4	92,206.66	46,102.82	4.7	1,619,403.56	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,174,198.13	980,502.06	104.6	854,165.63	389,676.27	35.1				705,496.53	Maryland
Massachusetts	14,047,656.22	6,467,661.28	300.6	792,371.07	275,356.11	20.7	6,030,673.96	1,587,507.67	80.1	1,161,802.28	286,284.20	16.9	2,492,916.74	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	8,225,220.35	3,783,676.07	301.6	10,230,087.14	4,641,089.56	262.1	382,294.00	178,607.13	17.2	4,410,775.33	Michigan
Minnesota	30,415,685.89	12,738,642.04	2,721.2	6,271,152.99	2,284,974.52	397.0	7,612,864.46	2,445,300.00	622.3	644,913.81	111,000.00	37.9	2,011,863.44	Minnesota
Mississippi	10,292,286.79	4,988,702.73	803.4	2,299,446.54	1,148,761.55	188.6	8,338,802.44	4,163,145.56	402.2	868,082.97	425,677.19	79.5	1,401,830.97	Mississippi
Missouri	17,368,166.57	8,219,411.43	1,118.9	6,738,800.74	2,782,790.50	228.2	23,931,410.82	9,467,301.10	669.7	1,632,876.60	666,671.48	47.6	1,650,361.39	Missouri
Montana	10,156,600.41	5,317,623.16	921.6	1,137,349.69	918,104.73	116.0	1,466,040.38	1,016,032.35	124.6	967,061.02	526,862.82	110.3	5,647,361.96	Montana
Nebraska	9,306,374.36	4,389,523.50	1,570.6	1,621,442.87	736,793.25	120.3	11,056,199.98	5,476,958.51	1,079.6	974,631.66	485,668.58	94.2	3,546,281.16	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	1,987,708.50	1,650,846.26	166.1	3,723,079.29	3,136,983.82	341.0				1,020,086.14	Nevada
New Hampshire	4,166,687.86	1,986,226.87	208.1	766,008.74	361,274.93	27.6	648,666.19	304,202.46	19.6	9,979.13	4,989.66	0.5	512,798.18	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	2,112,044.24	581,403.46	35.1	8,463,106.76	3,104,060.72	66.4				961,275.83	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	3,645,064.81	2,313,941.49	329.6	1,476,466.48	972,341.77	116.3	74,666.37	46,757.90	6.4	2,726,274.23	New Mexico
New York	28,697,769.57	12,229,076.53	831.6	8,334,912.28	3,326,763.27	210.2	31,966,396.58	9,664,379.74	604.0	8,652,570.00	2,046,840.00	136.8	6,879,135.46	New York
North Carolina	21,014,460.41	8,746,454.59	1,119.8	3,279,250.74	1,394,509.94	89.7	8,436,413.26	3,635,746.63	192.1	1,367,383.75	429,981.85	31.5	1,610,512.99	North Carolina
North Dakota	10,829,263.82	5,258,930.47	1,917.5	1,250,083.84	646,843.12	248.8	3,676,089.74	1,817,763.04	452.2	966,712.98	478,366.38	131.0	2,537,766.99	North Dakota
Ohio	41,572,252.81	16,244,993.93	1,191.1	6,893,936.99	2,030,223.35	167.4	8,626,466.69	3,384,262.39	268.6	2,066,699.69	669,907.97	57.5	4,402,418.36	Ohio
Oklahoma	20,787,024.94	9,672,890.34	862.2	3,088,439.52	1,427,381.27	131.0	6,446,480.58	3,078,587.94	285.3	234,268.05	110,766.51	7.3	1,770,161.94	Oklahoma
Oregon	14,388,168.70	7,142,364.63	794.5	1,771,199.12	928,700.66	84.1	2,633,017.61	1,683,066.23	135.4	1,567,212.09	296,739.58	46.9	928,485.91	Oregon
Pennsylvania	43,054,836.19	16,222,023.97	860.3	6,468,460.58	1,569,298.46	97.1	34,841,333.23	9,494,460.47	647.6	3,509,702.73	1,217,369.99	92.1	2,835,628.11	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,243,314.98	392,640.21	20.5	1,674,618.20	474,796.67	29.9				680,545.13	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.9	1,272,166.96	467,770.95	70.1	7,546,645.36	3,366,982.77	378.6	451,844.92	171,520.84	33.2	673,981.90	South Carolina
South Dakota	12,091,434.67	6,989,879.00	1,447.9	3,986,687.00	1,922,620.92	627.9	4,064,234.51	1,952,081.51	624.1				1,302,208.57	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	5,756,394.04	2,538,531.89	179.3	8,331,631.24	3,873,883.85	306.2	811,106.50	376,579.31	12.6	1,759,616.18	Tennessee
Texas	54,120,970.83	21,067,940.12	3,907.1	11,299,011.91	4,758,339.05	676.1	19,176,116.50	8,418,424.91	1,156.8	3,094,582.99	1,386,862.84	126.8	4,986,804.08	Texas
Utah	6,269,159.41	3,818,836.91	423.1	498,544.09	331,506.86	13.3	3,145,196.83	2,185,526.13	264.4	44,350.46	29,939.56	0.1	1,462,969.55	Utah
Vermont	3,016,174.61	1,462,894.45	107.8	995,612.97	463,868.72	21.4	1,275,735.50	549,360.08	28.4				812,383.75	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	7,144,835.58	3,308,996.46	268.3	5,289,885.13	2,300,376.91	168.4	3,003,893.02	1,343,816.12	76.8	276,327.32	Virginia
Washington	13,362,504.18	6,117,211.87	526.7	3,591,201.52	1,607,897.59	137.2	2,626,208.65	1,232,400.00	27.6				1,188,266.54	Washington
West Virginia	7;343,200.86	3,230,293.33	326.7	843,406.01	404,579.97	37.5	6,394,817.82	2,493,134.92	150.9	327,937.93	133,956.79	11.3	1,090,545.99	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1,451.7	1,467,080.92	722,678.76	67.6	6,484,949.02	2,694,686.42	261.6	335,059.18	167,528.00	16.6	4,934,381.20	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	1,774,399.61	1,080,637.38	145.4	2,714,091.06	1,706,666.53	166.2	28,580.20	18,348.00	0.1	1,022,525.42	Wyoming
Hawaii							342,277.22	97,440.00	6.6				1,002,713.00	Hawaii
TOTALS	740,140,790.82	326,654,346.00	41,898.3	141,841,633.53	64,450,912.34	7,067.0	361,925,907.24	153,872,464.12	14,731.1	41,893,608.12	14,992,448.81	1,740.1	112,404,828.73	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 104,884,653.56 Federal aid $ 44,450,778.49 Miles 3,928.0

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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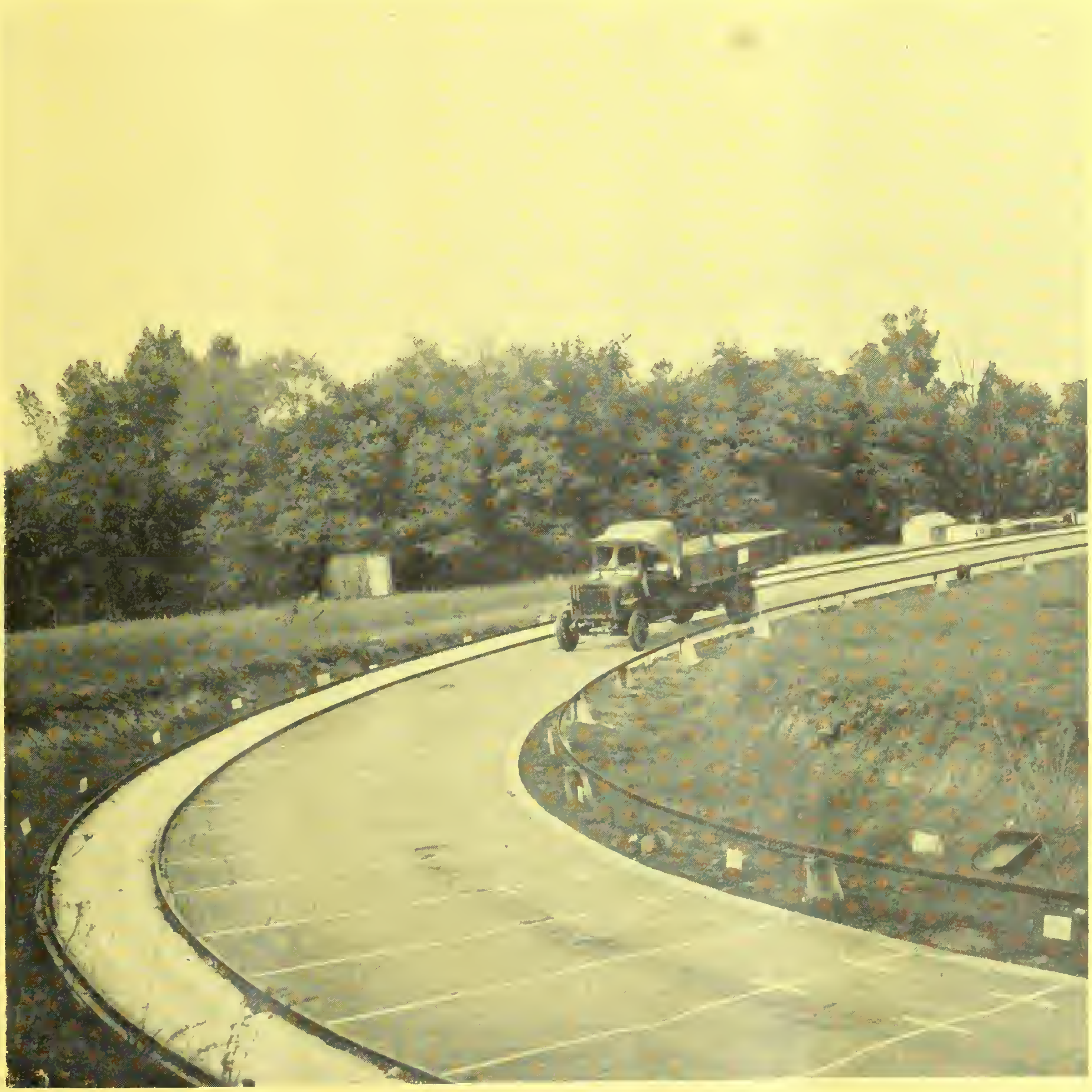

TESTING ASPHALTIC MIXTURES ON CIRCULAR TRACK AT ARLINGTON, VA.

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U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 7, NO. 2 APRIL, 1926 H. S. FAIRBANK, Editor

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STRESSES IN CONCRETE PAVEMENTS COMPUTED BY THEORETICAL ANALYSIS[1]

By **H. M. WESTERGAARD**, Associate Professor of Theoretical and Applied Machanics, University of Illinois

Doctor Westergaard, in the following paper prepared for presentation before the Highway Research Board, summarizes the results of a long period of study partly under the auspices of the Bureau of Public Roads. From assumptions of the conditions of loading, support, etc., of concrete road slabs, which conform closely to the actual conditions generally obtaining, he has developed by mathematical analysis a method by which the stresses in road slabs may be computed. By the use of the formulas, charts and tables which accompany the paper the method can be applied conveniently by highway engineers for the design of concrete road slabs. It also offers a means of computing the critical stresses in existing pavements, and may be used to furnish the answer to the question, often propounded, as to the possible decrease in the thickness of a pavement if the operation of the heavier vehicles is prohibited, and, vice versa, what additional thickness is required by a given increase in wheel pressure.

One may obtain a computation of stresses in concrete roads by assuming the slab to act as a homogeneous, isotropic, elastic solid in equilibrium, and by assuming the reactions of the subgrade to be vertical only and to be proportional to the deflections of the slab. With these assumptions introduced, the analysis is reduced to a problem of the mathematical theory of elasticity.

The reaction of the subgrade per unit of area at any given point may be expressed as a coefficient, k, times the deflection, z, at the point. This coefficient is a measure of the stiffness of the subgrade, and may be stated in pounds per square inch of area per inch of deflection, that is, in lb./in.^{3}. The coefficient, k, will be called the *modulus of subgrade reaction.* It corresponds to the "modulus of elasticity of rail support" which has been used in recent investigations of stresses in railroad track.[2] The modulus, k, is assumed to be constant at each point, independent of the deflections, and to be the same at all points within the area which is under consideration.

It is true that tests of bearing pressures on soils have indicated a modulus, k, which varies considerably depending upon the area over which the pressure is distributed.[3] Yet, so long as the loads are limited to a particular type, that of wheel loads on top of the pavement, it is reasonable to assume that some constant value of the modulus, k, determined empirically, will lead to a sufficiently accurate analysis of the deflections and the stresses. One finds an argument in favor of the assumption of a constant modulus, k, for a given section of road by examining the tables which are given below. They show that an increase of k from 50 lb./in.^{3} to 200 lb./in.^{3}, that is, an increase of the stiffness of the subgrade in the ratio of four to one, causes only minor changes of the important stresses. Minor variations of k, therefore, can be of no great consequence, and an approximate single value of k should be sufficient for a quite accurate determination of the important stresses within a given section of the road. The modulus, k, enters in the formulas for the deflections of the pavements, and may be determined empirically, accordingly, for a given type of subgrade, by comparing the deflections found by tests of full-sized slabs with the defections given by the formulas.

It will be assumed for the time being that the thickness of the slab is uniform and is equal to h.

A certain quantity which is a measure of the stiffness of the slab relative to that of the subgrade occurs repeatedly in the analysis. It is of the nature of a linear dimension, like, for example, the radius of gyration. It will be called the *radius of relative stiffness.* Denoted by l, it is expressed by the formula

$$l=\sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \quad \text{-------------} \quad (1)$$

where E is the modulus of elasticity of the concrete, and μ is Poisson's ratio of lateral expansion to longitudinal shortening. The stiffer the slab, and the less stiff the subgrade, the greater is l. One may observe that l remains constant when E and k are multiplied by the same ratio. Table 1 contains values of l for three different values of k and for different thicknesses of the slab. In computing this table as well as the three tables following, Poisson's ratio, μ, was assumed to be 0.15. This value agrees satisfactorily with the results of tests by A. N. Johnson.[4] The values of l given in the table lie between 16 and 55 inches, and about 36 inches may be considered to be a typical average.

THREE CASES OF LOADING INVESTIGATED

Figure 1 shows three cases in which it is of particular interest to be able to compute the critical stresses. In Case I a wheel load acts close to a rectangular corner of a large panel of the slab. This load tends to produce a corner break. The critical stress is a tension at the top of the slab. The resultant pressure is assumed to be on the bisector of the right angle of the corner at the small distance a from each of the two intersecting edges; the distance from the corner, accordingly is $a_1=a\sqrt{2}$. In Case II the wheel load is at a considerable distance from the edges. The pressure is assumed to be distributed uniformly over the area of a small circle with radius a. The critical tension occurs at the bottom of the slab under the center of the circle. In Case III the wheel load is at the edge, but at a considerable distance from any corner. The pressure is assumed to be distributed uniformly over the area of a small semicircle with the center at the edge and with radius a. The critical stress is a tension at the bottom under the center of the circle. In each of the three cases the load mentioned is assumed for the time being to be the only load acting.

---

[1] A paper presented before the annual meeting of the Highway Research Board, National Research Council, held at Washington, D. C., Dec. 3, 1925.

[2] Progress report of the special committee to report on stresses in railroad track, Am. Soc. Civil Engineers, Trans., v. 82, 1918, p. 1191.

[3] Tests dealing with this question have been reported in a paper entitled, "Researches on the structural design of highways by the United States Bureau of Public Roads," by A. T. Goldbeck, Am. Soc. Civil Engineers, Trans., v. 88, 1925, p. 264, especially p. 271; in a paper entitled, "The supporting value of soil as influenced by the bearing area," by A. T. Goldbeck and M. J. Bussard, PUBLIC ROADS, v. 5, No. 11, Jan. 1925; and by A. Bijls, in *Génie Civil*, v. 82, 1923, p. 490. According to these tests, in the case of a pressure which is distributed uniformly over an area, the modulus, k, would be approximately inversely proportional to the square root of the area. This result is supported by theoretical considerations.

[4] "Direct measurement of Poisson's ratio for concrete," by A. N. Johnson, Am. Soc. for Testing Materials, Proc., v. 24, Part II, 1924, p. 1024.

TABLE 1.—*Values of the radius of relative stiffness, l, for different values of the slab thickness, h, and of the modulus of subgrade reaction, k, computed from equation (1)*

E=3,000,000 pounds per square inch. μ=0.15

Thickness of slab in inches h	Radius of relative stiffness, l, in inches		
	k=50 lb./in.^{3}	k=100 lb./in.^{3}	k=200 lb./in.^{3}
4	23. 91	20. 11	16. 92
5	28. 28	23. 78	20. 00
6	32. 40	27. 26	22. 92
7	36. 40	30. 60	25. 73
8	40. 23	33. 83	28. 44
9	43. 94	36. 95	31. 07
10	47. 55	40. 00	33. 62
11	51. 08	42. 94	36. 11
12	54. 52	45. 84	38. 56

For Case I a computation which may be looked upon as a first approximation was proposed by A. T. Goldbeck. Further emphasis was given to this method by Clifford Older.[5] The load is treated as a force concentrated at the corner itself—that is, one assumes $a=a_1=0$. At small distances from the corner the influence of the reactions of the subgrade upon the stresses will be small compared with that due to the load. The corner portion may be considered, therefore, to act as a cantilever of uniform strength. At the distance x, measured diagonally from the corner along the bisector of the right angle of the corner, the bending moment is $-Px$. This bending moment may be assumed to be distributed uniformly over the cross section, the width of which is $2x$. Thus one finds the bending moment per unit of width of cross section equal to $-\frac{P}{2}$, and the tensile stress at the top equal to

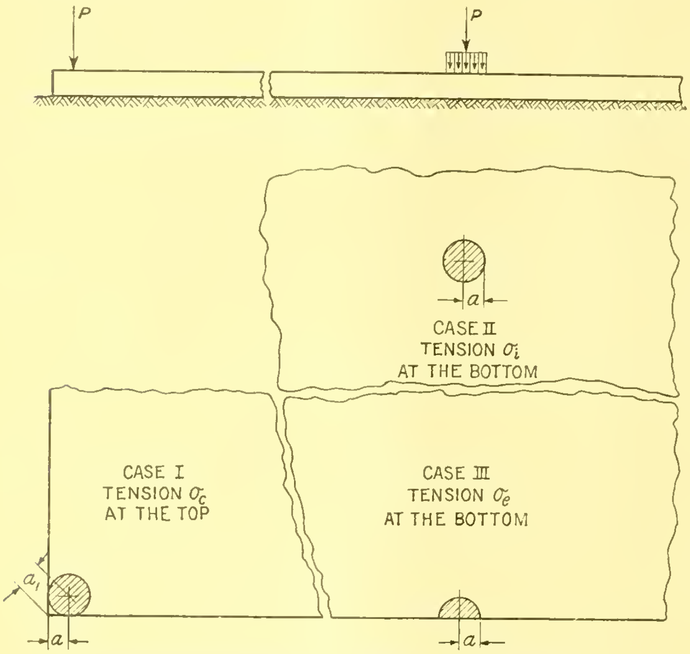

FIG. 1.—Three cases of loading. The corresponding greatest stresses are given in Tables 2 to 4

$$\sigma=\frac{3P}{h^2} \text{ - } (2)$$

Since the wheel load is distributed over the area of contact between the tire and the pavement, the distances a and a_1 can not be zero. The greatest stress occurs, then, at some distance from the load. This distance will be sufficiently large to make the reactions of the subgrade outside the critical section contribute a noticeable reduction of the numerical value of the bending moment.

An improved approximation has been obtained in the following manner. The origin of the horizontal rectangular coordinates x and y is taken at the corner, the axis of x bisecting the right angle of the corner. By use of Ritz's method of successive approximation, which is based on the principle of minimum of energy,[6] the following approximate expression was found for the deflections in the neighborhood of the corner:

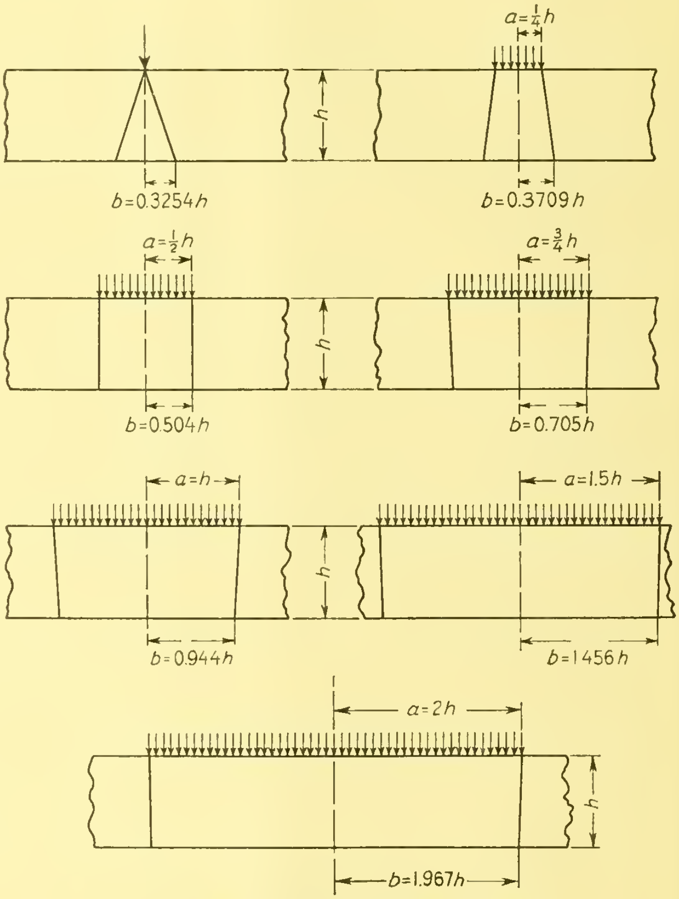

FIG. 2.—Cones of equivalent distribution of pressure

$$z=\frac{P}{kl^2}\left(1.1e^{-\frac{x}{l}}-\frac{a_1}{l}\ 0.88e^{-\frac{2x}{l}}\right) \text{ - - - - - - - - - - - - - - } (3)$$

Then the reactions of the subgrade will be expressed with sufficient exactness in terms of this function as kz. One may compute, then, the total bending moment, M', in the section $x=x_1$ due to the combined influence of the applied load and the reactions of the subgrade. When x_1 is not too large, this bending moment will be approximately uniformly distributed over the width, $2x_1$, of the cross section. That is, the bending moment per unit of width becomes $M=\frac{M'}{2x_1}$. The numerically greatest value of M was found, in this manner, to occur approximately at the distance

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[5] "Highway research in Illinois," by Clifford Older, Am. Soc. Civil Engineers, Trans., v. 87, 1924, p. 1180, especially p. 1206.

[6] W. Ritz, Crelle's Journal, v. 135, 1909, p. 1.

$$x_1 = 2\sqrt{a_1 l} \qquad (4)$$

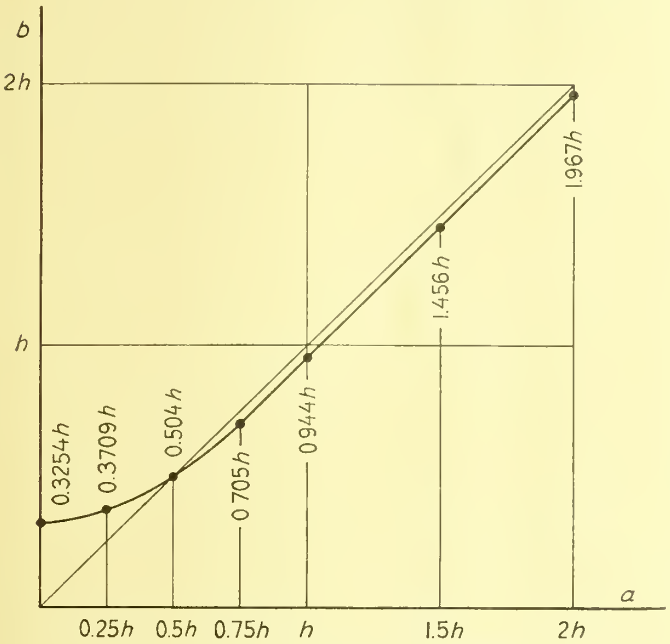

FIG. 3.—Relation between the true radius, a, the equivalent radius, b, and the thickness, h

and to be, approximately,

$$M = -\frac{P}{2}\left[1-\left(\frac{a_1}{l}\right)^{0.6}\right] \qquad (5)$$

Division by the section modulus per unit of width, $\frac{h^2}{6}$, leads to the corresponding greatest tensile stress

$$\sigma_c = \frac{3P}{h^2}\left[1-\left(\frac{a_1}{l}\right)^{0.6}\right] \qquad (6)$$

This stress may be stated also in the following form which is derived by substituting the value of l from equation (1):

$$\sigma_c = \frac{3P}{h^2}\left[1-\left(\frac{Eh^3}{12\,(1-\mu^2)\,k}\right)^{-0.15} a_1^{0.6}\right] \qquad (7)$$

With $a_1 = 0$, the last two equations assume the simpler form of equation (2).

STRESS NOT GREATLY AFFECTED BY SUBGRADE CONDITION

Table 2 contains numerical values of the critical stress σ_c for $P = 10{,}000$ pounds, $E = 3{,}000{,}000$ pounds per square inch, and $\mu = 0.15$. The table shows the influence of three variables: The thickness, h; the modulus of subgrade reaction, k; and the distance, a, from the edges to the center of the load.

An inspection of the table shows the influence of the variation of the distance, a, to be appreciable, amounting easily to a reduction of more than 30 per cent as compared with the value found by the first approximation, with $a = 0$. The influence of the variation of the modulus, k, from 50 to 200 lb./in.³, on the other hand, is not particularly large.

In Case II, that of a wheel load at a point of the interior, complications arise due to the fact that the load is concentrated within a rather small area. The theory of elasticity offers two types of theory of slabs: One that may be called the "ordinary theory of slabs," and the other, the "special theory." The difference may be explained by an analogy with beams. In analysis of beams it is assumed ordinarily that a plane cross section remains plane and perpendicular to the neutral surface during the bending. For beams of ordinary proportions, this assumption leads to satisfactory results, unless one is concerned with the local stresses in the immediate neighborhood of a concentrated load. In the latter case the assumption of the plane cross section must be abandoned, and a special theory, which takes into account the deformations due to the vertical stresses, is required. In the ordinary theory of slabs it is assumed, correspondingly,

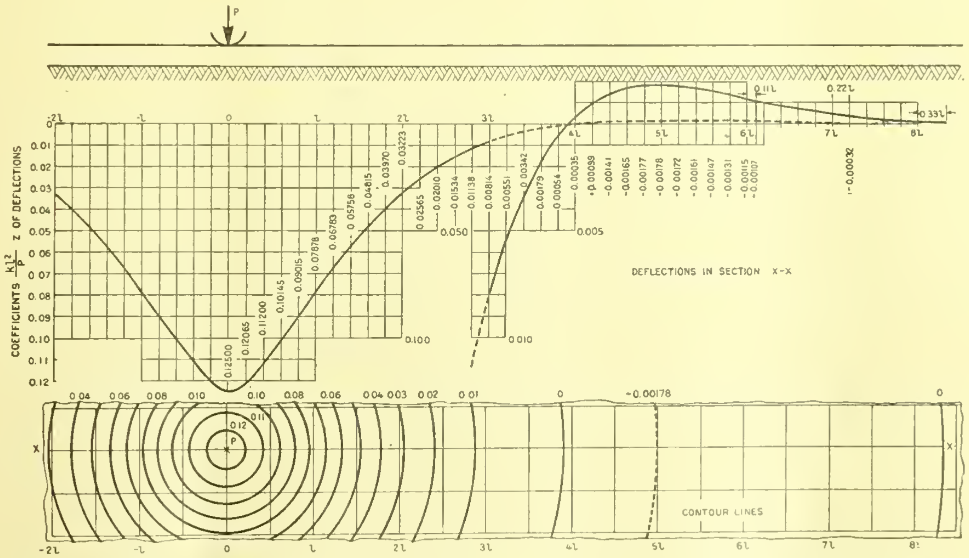

FIG. 4.—Deflections produced by a concentrated load which acts at a point of the interior at a considerable distance from the edges

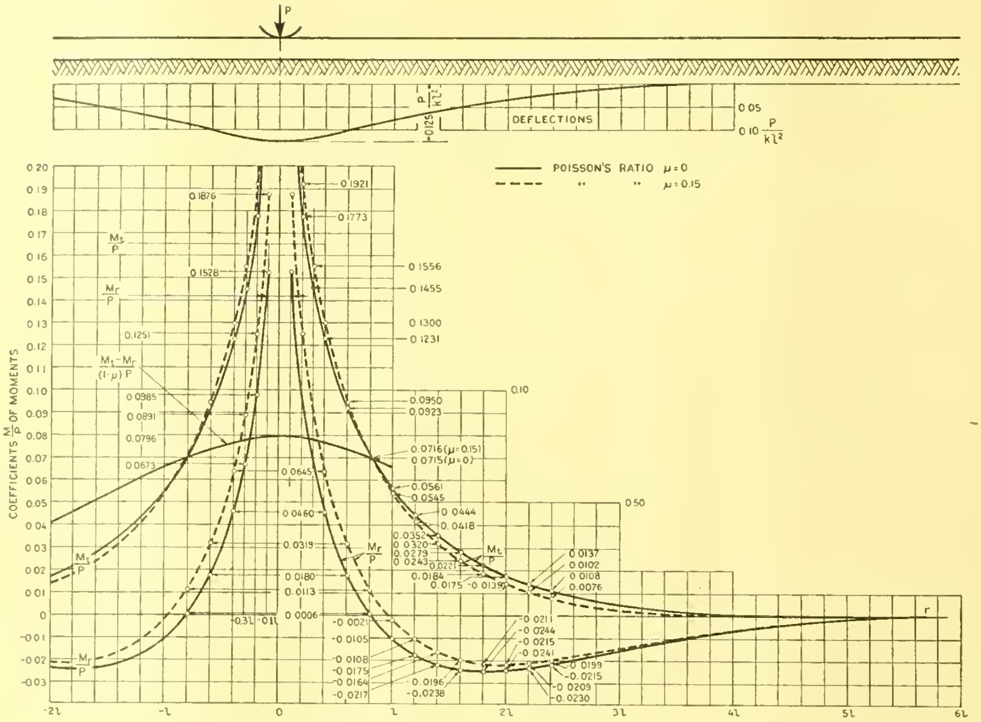

Fig. 5.—Tangential bending moments, M_t, and radial bending moments, M_r, produced by a concentrated load which acts at a point of the interior at a considerable distance from the edges

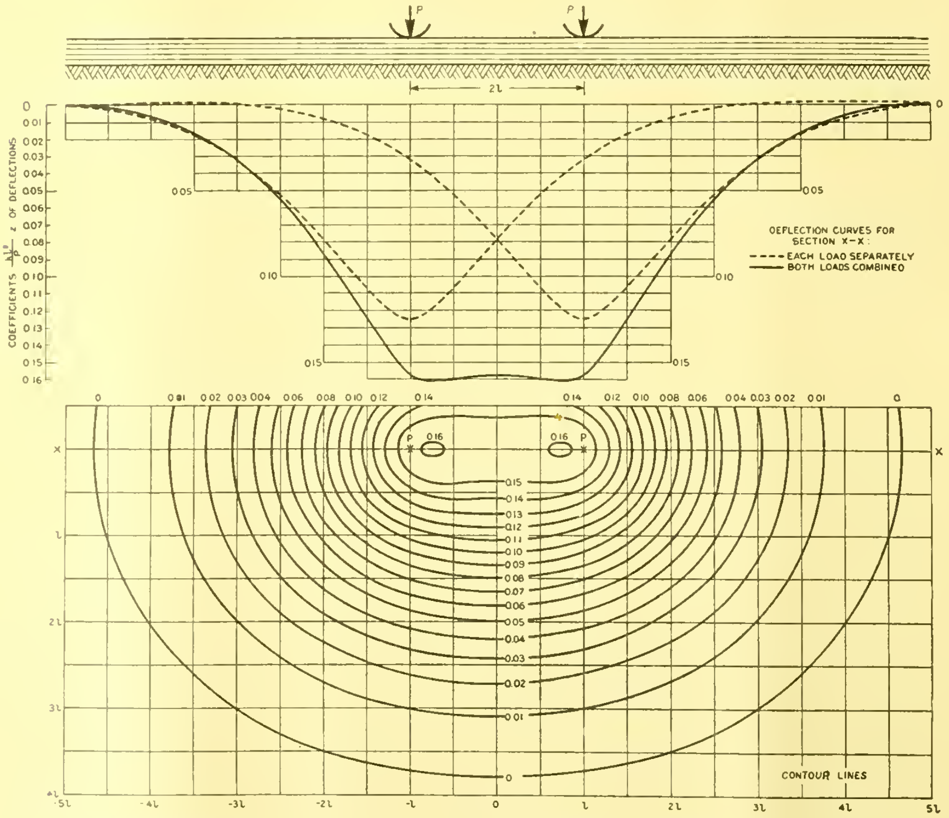

Fig. 6.—Deflections produced by two equal loads like the load in Figure 4, separated by a distance of 2l. The deflections are found by superposition of two diagrams of the kind shown in Figure 4

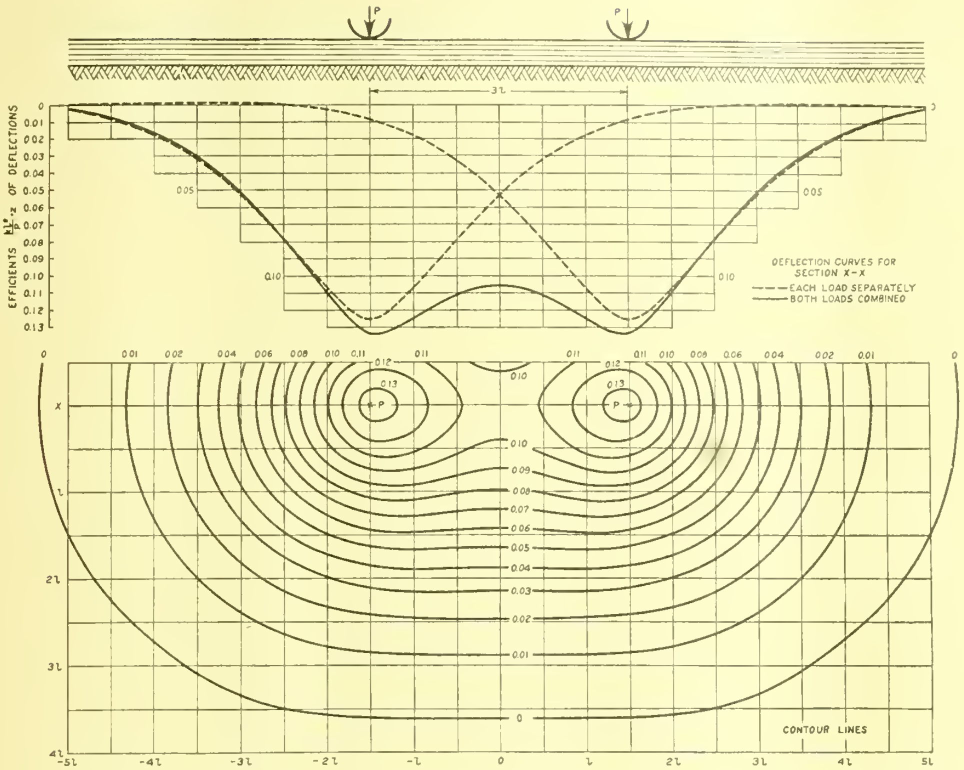

FIG. 7.—Deflections produced by two equal loads like the load in Figure 4, separated by a distance of 3l

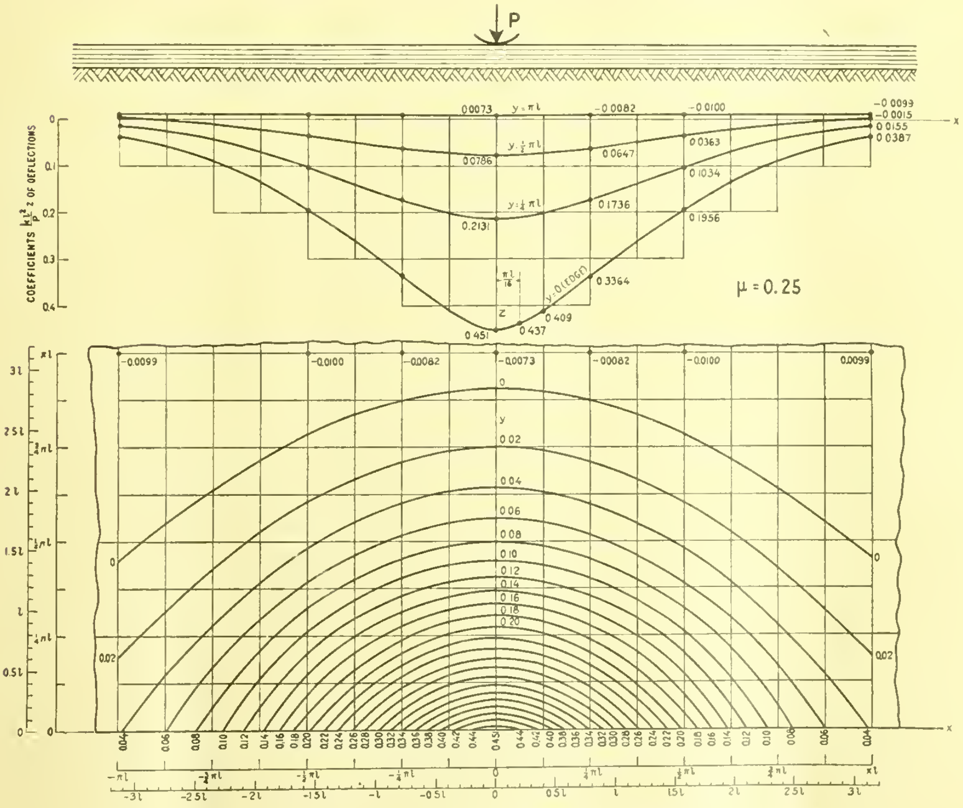

FIG. 8.—Deflections produced by a concentrated load at the edge at a considerable distance from any corner for $\mu=0.25$

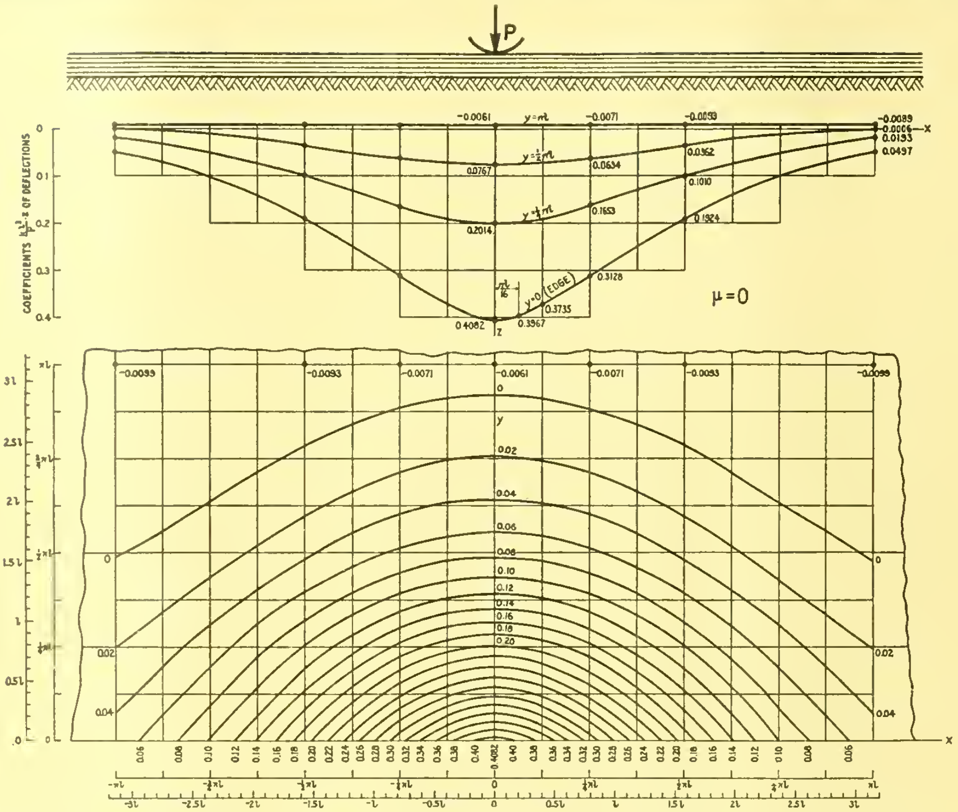

FIG. 9.—Deflections produced by a concentrated load at the edge at a considerable distance from any corner for μ=0

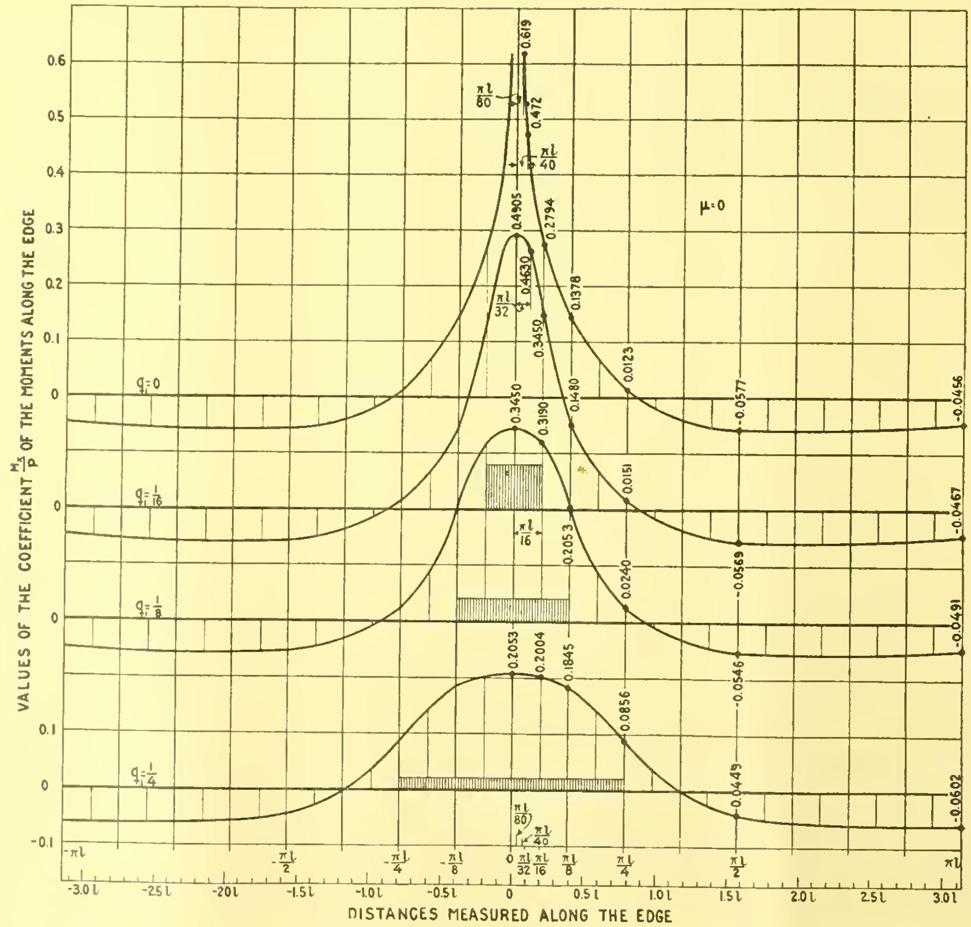

FIG. 10.—Bending moments along the edge for a load concentrated at a point of the edge (top diagram), and for loads distributed uniformly over lines of three lengths at the edge (lower three diagrams); μ=0

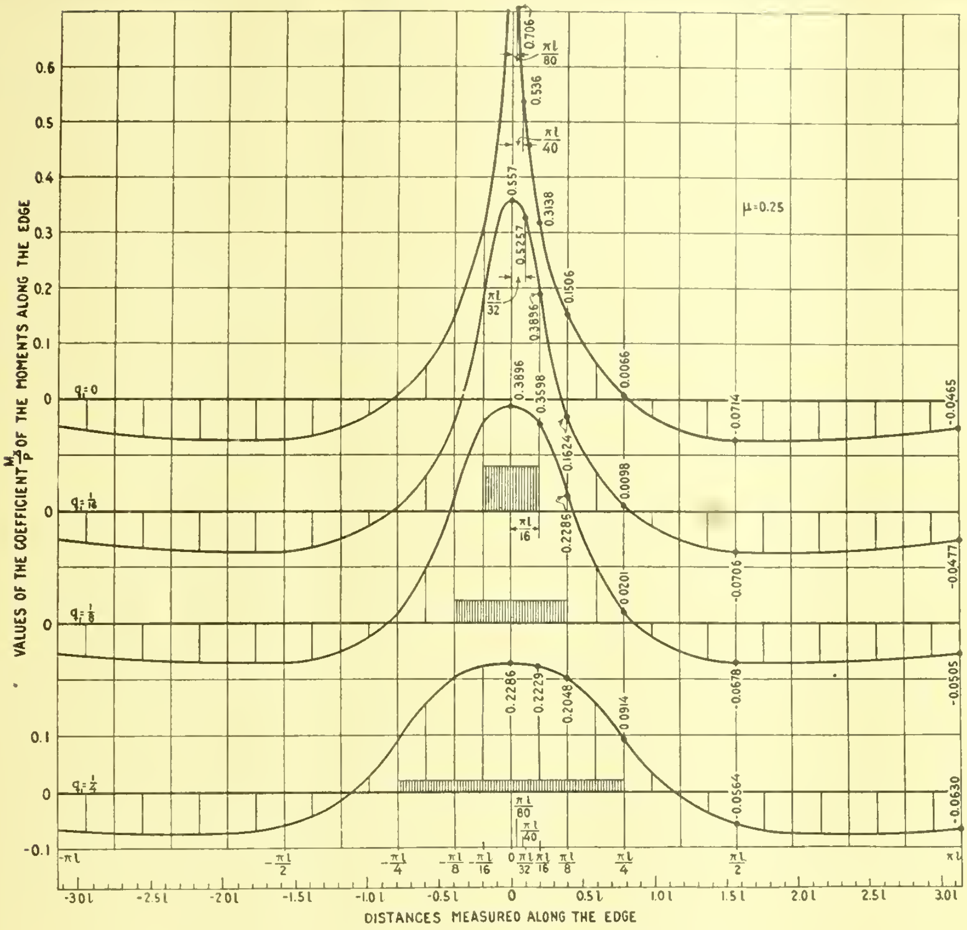

FIG. 11.—Bending moments along edge as in Figure 10, but for $\mu = 0.25$

that a straight line drawn through the slab perpendicular to the slab remains straight and perpendicular to the neutral surface.

TABLE 2.—*Stresses in pounds per square inch computed from equation (7) for load condition as in Case I, Figure 1, for different values of h, k, and a*

P=10,000 pounds, E=3,000,000 pounds per square inch, μ=0.15

Thickness of slab, *h*	Modulus of subgrade reaction, *k*	Stress in slab			
		a=0	*a*= 2 inches	*a*= 4 inches	*a*= 6 inches
Inches	*Lb./in.*³	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
6	50	833	641	541	461
	100	833	619	509	420
	200	833	596	474	375
7	50	612	480	412	357
	100	612	466	390	329
	200	612	450	366	298
8	50	469	373	325	285
	100	469	363	309	265
	200	469	352	291	242
9	50	370	299	262	233
	100	370	291	250	217
	200	370	282	237	201
10	50	300	245	216	193
	100	300	239	207	182
	200	300	232	197	169
11	50	248	204	182	164
	100	248	200	175	154
	200	248	194	167	144
12	50	208	173	155	140
	100	208	169	149	133
	200	208	165	143	124

With slabs of proportions as found in pavements, the theory based on these assumptions leads to a satisfactory determination of stresses at all points except in the immediate neighborhood of a concentrated load, and leads to a satisfactory determination of the deflections at all points. At the point of application of a concentrated force this ordinary theory leads to a peak in the diagrams of bending moments, with infinite values at the point of the load itself (as indicated in figs. 5, 10, and 11). When the force is applied at the top of the slab, the tensile stresses at the bottom are not, in fact, infinite. One may say then that the effect of the thickness of the slab is equivalent to a rounding off of the peak in the diagrams of moments. In order to find out to what extent the diagrams are rounded off, it is necessary to abandon the assumption of the straight lines drawn through the slab remaining straight, as applying to the immediate neighborhood of the load, and a special theory is required. This special theory rests on only two assumptions: One is that Hooke's law applies, the constants being the modulus of elasticity, E, and Poisson's ratio, μ; the other is that the material keeps its geometrical continuity at all points. As in the case of beams, the ordinary theory is much simpler than the special theory, and is used, therefore, except in particular cases like the present one, which deals with local effects around a concentrated load.

It is expedient to express the results of the special theory in terms of the ordinary theory in the following manner: Let the load, P, be distributed uniformly over the area of the small circle with radius a. The tensile stress produced by this load at the bottom of the slab under the center of the circle is denoted by σ_i. This stress is the critical stress except when the radius, a, is so small that some of the vertical stresses near the top become more important; the latter exception need not be considered, however, in case of a wheel load which is applied through a rubber tire. By use of the ordinary theory one may find the same stress at the same place by assuming the load to be distributed over the area of a circle with the same center, but with the radius b. One finds that this equivalent radius, b, can be expressed with satisfactory approximation in terms of the true radius, a, and the thickness, h, only.

In order to find the relation between h, a, and b, numerical computations were made in accordance with an analysis which is due to A. Nádai.[7] The center of the load P is assumed for the time being to be at the center of a circular slab. The slab is supported at the edge in such a manner that the sum of the radial and tangential bending moments is zero at every point of the edge. Computations according to Nádai's analysis, with the radius of the slab equal to $5h$, gave the results which are represented in Figure 2 in the manner of "cones of equivalent distribution" and in Figure 3 by a curve with coordinates a and b. Approximately the same cones and the same curve are obtained for other radii of the slab; and the results may be applied generally to slabs of proportions such as are found in concrete pavements, with any kind of support which is not concentrated within a small area close to the load.

One may notice that when a increases gradually from zero, b is at first larger than a; but when a passes a certain limit, b becomes smaller than a. For the larger values of a, the ratio, $\frac{b}{a}$, converges toward unity, and the ordinary theory of slabs, accordingly, gives nearly the same results as the special theory.

The curve in Figure 3 is found to lie close to a hyperbola, the equation of which may be written in the following form, which is suitable for numerical computations, and which may be used for values of a less than $1.724h$:

$$b=\sqrt{1.6a^2+h^2}-0.675h \qquad \text{(8)}$$

For larger values of a, one may use $b=a$, that is, the ordinary theory may be used without corrections.

By the ordinary theory one finds the following approximate expression for the critical stress:

$$\sigma_i=\frac{3(1+\mu)}{2\pi h^2}P\left(\log_e\frac{l}{a}+0.6159\right) \qquad \text{(9)}$$

With $E=3{,}000{,}000$ pounds per square inch and $\mu=0.15$, and with l substituted from equation (1), this formula takes the form:

$$\sigma_i=0.3162\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}a-\log_{10}k+6.478] \qquad \text{(10)}$$

The correction to be made in this formula in order to make it agree with the special theory is merely to replace the true radius, a, by the equivalent radius, b. Thus one finds the following formula, which replaces equation (10) when a is less than $1.724h$:

$$\sigma_i=0.3162\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}(\sqrt{1.6a^2+h^2}-0.675h)-\log_{10}k+6.478] \qquad \text{(11)}$$

The stresses given in Table 3 have been computed in accordance with this formula for $P=10{,}000$ pounds. Like Table 2, this table shows the influence of three variables: the thickness, h; the modulus of subgrade reaction, k, and a. In Table 3, as in Table 2, one may notice the relatively greater influence of the variation of a as compared with the influence of the variation of k.

In dealing with Case III, that of a wheel load at the edge, it was assumed that an equivalent radius, b, may be introduced in the place of the true radius, a, in the same manner as in the preceding case, and by the same formula, that of equation (8). This assumption may be justified on the ground of the similarity in the two cases in the distribution of the energy due to vertical shearing stresses. By introducing the equivalent radius, b, in the place of a in the formula for the tensile stress, σ_e, along the bottom of the edge under the center of the circle, as obtained by the ordinary theory, one finds the following expression which, like the analogous equation (11), is based on $E=3{,}000{,}000$ pounds per square inch and $\mu=0.15$:

$$\sigma_e=0.572\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}(\sqrt{1.6a^2+h^2}-0.675\,h)-\log_{10}k+5.767] \qquad \text{(12)}$$

Stresses computed according to this formula are given in Table 4, again for $P=10{,}000$ pounds. The influence of the three variables, h, k, and a, is shown in the same manner as in the two preceding tables, and is seen to be of the same nature, the variation of a being of greater importance than that of k.

TABLE 3.—*Stresses in pounds per square inch computed from equation (11) for load condition as in Case II, Figure 1, for different values of h, k, and a*

P=10,000 pounds, E=3,000,000 pounds per square inch, μ=0.15

Thickness of slab h	Modulus of subgrade reaction k	Stress in slab				
		a=0	a=2 in.	a=4 in.	a=6 in.	a=8 in.
Inches	*Lb./in.³*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
4	50	1,231	1,058	848	693	588
	100	1,172	998	788	634	528
	200	1,112	939	729	574	469
5	50	763	694	580	487	415
	100	725	656	542	449	377
	200	687	617	504	411	339
6	50	523	487	421	361	313
	100	497	461	395	335	287
	200	470	435	368	308	260
7	50	380	360	319	279	245
	100	361	341	300	260	226
	200	341	321	280	240	206
8	50	288	276	250	222	197
	100	273	261	235	207	182
	200	258	246	220	192	167
9	50	226	218	200	180	162
	100	214	206	188	169	150
	200	202	194	177	157	138
10	50	181	176	164	149	136
	100	172	167	154	140	126
	200	162	157	145	130	116

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[7] A. Nádai, See "Die Biegungsbeanspruchung von Platten durch Einzelkräfte," Schweizerische Bauzeitung, v. 76, 1920, p. 257; and his book, "Die elastischen Platten," (Berlin) 1925, p. 308.

TABLE 4.—*Stresses in pounds per square inch computed from equation (12), for load condition as in Case III, Figure 1, for different values of h, k, and a*

P=10,000 pounds, E=3,000,000 pounds per square inch, μ=0.15

Thickness of slab h	Modulus of subgrade reaction k	Stress in slab				
		a=0	a=2 in.	a=4 in.	a=6 in.	a=8 in.
Inches	*Lb./in.*^{3}	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
6	50	833	769	649	541	453
	100	785	721	601	493	406
	200	738	673	553	445	358
7	50	604	568	494	422	360
	100	569	533	459	386	325
	200	534	498	424	351	290
8	50	457	436	388	337	293
	100	430	409	361	311	266
	200	404	382	334	284	239
9	50	358	344	312	276	243
	100	337	323	291	255	222
	200	315	301	269	233	200
10	50	287	278	256	230	204
	100	270	261	239	212	187
	200	253	244	221	195	170
11	50	235	229	213	194	174
	100	221	215	199	180	160
	200	207	201	185	165	146
12	50	196	192	180	165	150
	100	184	180	168	153	138
	200	172	168	156	142	126

BALANCED DESIGNS TESTED BY USE OF TABLES

From the three tables, for Cases I, II, and III, one may obtain suggestions on the question of balanced design. Consider, for example, a pavement with the thickness of 7 inches in the interior portion, and 9 inches at the edges. It may be assumed for the time being that the outer portions behave as a large slab with uniform thickness of 9 inches. With the thickness diminishing slowly toward the interior, the stresses σ_c and σ_e would be somewhat larger than with constant thickness of 9 inches, but the correction needed for this reason is probably only small. For the time being only the one wheel load which is considered in each of the three tables will be taken into account. The influence of other wheel loads acting on the same panel, but at some distance, will be considered later; in any case it is found to be relatively small. With $P=10{,}000$ pounds, $k=50$ lb./in.^{3}, and $a=4$ inches, the three tables give the following values:

$$\sigma_c=262 \text{ lb. per sq. in., } \sigma_i=319 \text{ lb. per sq. in., } \sigma_e=312 \text{ lb. per sq. in.}$$

In comparing these stresses, their different characters should be considered. The stress, σ_c, at the corner acts presumably throughout the width of a whole cross section, whereas σ_i and σ_e are localized within smaller regions. With equal tendency to rupture at the three places, σ_c then, should be, probably, somewhat smaller than σ_i and σ_e. The stress, σ_e, is produced under the influence of a load which is distributed over an area only one-half of that assumed for σ_i. Although the situation represented by the smaller area may occur when a wheel moves in over the edge of the pavement, it is reasonable, for the purpose of a comparative study of the tendency to rupture, to assume a larger radius of the semi-circle at the edge than for the full circle in the interior portion. With $a=6$ inches, for example, at the edge, one finds the stress

$$\sigma_e=276 \text{ lb. per. sq. in.}$$

In comparing this stress with σ_i it should be observed that σ_i represents a state of equal stresses in all horizontal directions at the point, whereas σ_e is a one-directional stress. The elongations per unit of length are in the two cases $\frac{\sigma_i(1-\mu)}{E}$ and $\frac{\sigma_e}{E}$. It appears to be reasonable, therefore, for the purpose of comparison, to replace σ_i by an equivalent one-directional stress; if in this case the elongation is a direct measure of the tendency to rupture, this equivalent stress should be

$$\sigma_i'=\sigma_i\ (1-\mu)=319\ (1-0.15)=271 \text{ lb. per sq. in.}$$

The three values 262, 271, and 276 pounds per square inch point toward the conclusion that the assumed design is suitably balanced.

The suggestion has been made already that one may determine suitable values of k by comparing the deflections found by tests of full-sized slabs with those given by the formulas. The following formulas lend themselves to this purpose; they refer to the three cases shown in Figure 1, and in each case the load P is the only one acting:

Case I. Equation (3) gives the deflection at the corner:

$$z_c=\left(1.1-0.88\,\frac{a_1}{l}\right)\frac{P}{kl^2} \quad \text{(13)}$$

Case II. The deflection under the center of the load differs only slightly from the following value which is accurate when $a=0$:

$$z_i=\frac{P}{8kl^2} \quad \text{(14)}$$

Case III. The deflection at the point of application of a concentrated force P at the edge is approximately equal to

$$z_e=\frac{1}{\sqrt{6}}\,(1+0.4\,\mu)\,\frac{P}{kl^2} \quad \text{(15)}$$

that is, for $\mu=0.15$,

$$z_e=0.433\,\frac{P}{kl^2} \quad \text{(16)}$$

The quantity kl^2 occurring in each of these formulas may be expressed, according to equation (1), as

$$kl^2=\sqrt{\frac{Eh^3k}{12(1-\mu^2)}} \quad \text{(17)}$$

When experimental values of the deflections are at hand, one may determine the corresponding values of kl^2 by means of equations (13) to (16). Then equation (17) gives the value of k as

$$k=\frac{12(1-\mu^2)(kl^2)^2}{Eh^3} \quad \text{(18)}$$

Figures 4 to 11 are diagrams of deflections and moments. The titles of these figures explain the nature of the diagrams. The deflections and bending moments have been computed by means of the ordinary theory of slabs. The diagrams, therefore, give information concerning deflections in general, and concerning bending moments except in the immediate neighborhood of the concentrated load which produces the bending moments.

The diagrams in Figure 4 and Figure 5 have been obtained by an analysis which rests essentially on that given by the physicist Hertz[8] in 1884.

DETERMINATION OF DEFLECTIONS DUE TO MORE THAN ONE WHEEL

The diagrams in Figures 4 and 5 may be used in the following way for the purpose of finding the resultant deflections and stresses due to the combined influence of two or four wheel loads, each acting at a considerable distance from the edges of the slab.

Let each load be 10,000 pounds and let the horizontal rectangular coordinates of the centers of the four loads be as follows:

Coordinate	Load No. 1	Load No. 2	Load No. 3	Load No. 4
$x=$	0	66 in.	0	66 in.
$y=$	0	0	66 in.	66 in.

Loads 1 and 2 alone may represent the two rear wheels of a four-wheel truck, and the four loads combined may represent the four rear wheels of a six-wheel truck.

With $h=7$ inches, $E=3{,}000{,}000$ pounds per square inch, $\mu=0.15$, and $k=50$ lb./in.^{3}, one finds by equations (1) and (17), or by Table 1:

$l=36.40$ inches; $kl^2=66{,}200$ pounds per inch; distances 1–2 and 1–3: 66 in. $=1.813l$; distance 1–4: $66\sqrt{2}=2.564l$. Then equation (14) as well as Figure 4 gives the following value of the deflection at point 1 due to Load No. 1:

$$z_{1,1}=\frac{P}{8kl^2}=\frac{10{,}000}{8\times 66{,}200}=0.0189 \text{ inch.}$$

Furthermore, Figure 4 leads to the following value of the deflection at point 1 due to load No. 2 alone:

$$z_{1,2}=0.03921\frac{P}{kl^2}=0.03921\frac{10{,}000}{66{,}200}=0.0059 \text{ inch.}$$

Then, by superposition of the two deflections, one finds the deflection at point 1 due to the combined influence of the two rear wheels 1 and 2:

$$z_{1,(1,2)}=z_{1,1}+z_{1,2}=0.0248 \text{ inch.}$$

The deflection at point 1 due to load No. 3 alone is

$$z_{1,3}=z_{1,2}=0.0059 \text{ inch.}$$

The deflection at point 1 due to load No. 4 alone is, according to Figure 4:

$$z_{1,4}=0.01620\frac{P}{kl^2}=0.0024 \text{ inch.}$$

By superposition of the four deflections due to each separate load, one finds the resultant deflection due to the four loads:

$$z_{1,(1,2,3,4)}=0.0331 \text{ inch.}$$

For the purpose of computing the state of stresses at the bottom of the slab under the center of load No. 1, it will be assumed that load No. 1 is distributed uniformly over the area of a circle with radius $a=6$ inches. The stresses due to load No. 1 will be the same in all directions, and they are, according to Table 3:

$$\sigma_x=\sigma_y=279 \text{ pounds per square inch.}$$

According to Figure 5, load No. 2 produces a radial bending moment, M_r, in this case in the direction of x, equal to $M_x=-0.0211P=-211$ inch-pounds per inch (or -211 pounds), and a tangential bending moment M_t, in this case in the direction of y, equal to

$$M_y=0.0181P=181 \text{ pounds.}$$

The corresponding stresses are found by dividing these bending moments by the section modulus per unit of width, that is, by $\frac{1}{6}h^2=8.167$ in.^{2}. Thus one finds the stresses in the directions of x and y:

$$\sigma_x=-\frac{211}{8.167}=-26 \text{ pounds per square inch}$$

and

$$\sigma_y=\frac{181}{8.167}=22 \text{ pounds per square inch.}$$

These stresses are principal stresses, that is, one is the maximum, the other the minimum stress, and there are no shearing stresses in the directions of x and y.

For the case of the four-wheel truck, one finds, then, by superposition, the following principal stresses due to the two rear wheels, loads No. 1 and No. 2, these principal stresses being in the directions of x and y:

$$\sigma_x=279-26=253 \text{ pounds per square inch.}$$

$$\sigma_y=279+22=301 \text{ pounds per square inch.}$$

STRESSES DUE TO SIX-WHEEL TRUCK

In the case of the six-wheel truck the effects of loads No. 3 and No. 4 must be included. Load No. 3 contributes the same stresses at point 1 as does load No. 2, only the indices x and y are to be interchanged. Consequently the resultant stresses in the directions of x and y due to the combined influence of loads 1, 2, and 3 become

$$\sigma_x=\sigma_y=279-26+22=275 \text{ pounds per square inch.}$$

These stresses, again, are principal stresses. Since they are equal, the horizontal stresses will be the same in all directions, each stress being a principal stress.

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[8] H. Hertz. See "Über das Gleichgewicht schwimmender elastischer Platten," Wiedemann's Annalen der Physik und Chemie, v. 22, 1884, pp. 449–455; also in his Gesammelte Werke, v. 1, pp. 288–294. Hertz dealt with the problem of a large swimming slab, for example, of ice, loaded by a single force. A. Föppl, in his Technische Mechanik, v. 5, 1907, pp. 112–130, presented Hertz's theory in a modified, and in some ways simplified form, and he called attention to the applicability of this analysis to the problem of the slab on elastic support. Hertz made use of Bessel functions in his analysis. Since his analysis was published, the number of published numerical tables of Bessel functions has been increased. Among the newer tables those representing Hankel's Bessel functions,

$$H_0^{(1)}(x\sqrt{i}) \text{ and } H_1^{(1)}(x\sqrt{i}),$$

are of especial interest for the present problem. Tables of these functions may be found in the book of tables by E. Jahnke and F. Emde, "Funktionentafeln mit Formeln und Kurven," 1909, pp. 139 and 140. By means of these tables the numerical values given in Figures 4 and 5 were obtained by simple computations. Since these diagrams were prepared, two papers have appeared in which the same functions are used for the purpose of analysis of slabs on elastic support. One is by J. J. Koch, "Berekening van vlakke platen, ondersteund in de hoekpunten van een willekeurig rooster," De Ingenieur, 1925, No. 6; the other is by Ferdinand Schleicher, "Über Kreisplatten auf elastischer Unterlage," Festschrift zur Hundertjahrfeier der Technischen Hochschule Karlsruhe, 1925.

Let x', y' be a new system of horizontal rectangular coordinates with the axis of x' along the diagonal line from point 1 to point 4. Load No. 4 produces a radial bending moment in the direction of x' and a tangential bending moment in the direction of y'. According to Figure 5 these bending moments are

$M_{x'} = -0.0186P = -186$ pounds and $M_{y'} = 0.0058P = 58$, pounds respectively. The corresponding stresses are found, again, by dividing the bending moments by the section modulus per unit of width, that is, by 8.167 in.^{2}, and they are

$\sigma_{x'} = -23$ pounds per square inch and $\sigma_{y'} = 7$ pounds per square inch.

These stresses are principal stresses. The resultant principal stresses due to all four loads combined, therefore, are in the directions of x' and y', and have the values

$$\sigma_{x'} = 275 - 23 = 252 \text{ pounds per square inch.}$$

$$\sigma_{y'} = 275 + 7 = 282 \text{ pounds per square inch.}$$

One may draw the conclusion that the main part of the state of stresses at a given point is due to a wheel load right over the point. In the case examined, the contribution due to the three additional rear wheels of the six-wheel truck is of less importance than that due to the one additional rear wheel of the four-wheel truck.

Figure 6 and Figure 7 show deflections due to two wheel-loads combined. Each of these diagrams was obtained by superposition of two diagrams such as shown in Figure 4.

Figures 8 to 11 show effects of loads at the edge, but at a considerable distance from any corner.[9]

By virtue of Maxwell's theorem of reciprocal deflections, the deflection at a point B of any slab due to a load P at the point A is the same as the deflection at A due to a load P at point B. Figures 8 and 9 may be interpreted, therefore, in a double manner: First, as diagrams of deflections at any point B due to a load P at the particular point A at the edge; secondly, as influence diagrams, showing the deflection at the particular point A at the edge due to a load P at any point.

From this reciprocity of deflections one may draw a further conclusion which may be applied to Figures 8 and 9, and which concerns the curve of deflections, or elastic curve, which is obtained by intersection of the deflected middle surface by a vertical plane. Two lines L_A and L_B are drawn parallel to two opposite parallel edges of a slab. Two equal loads are considered, one acting at a point A of the line L_A, the other acting at a point B of the line L_B. The points A and B are assumed to be sufficiently far from the remaining two edges of the slab to permit the assumption of zero deformations at these edges. Then one may conclude that the elastic curve produced along the line L_B under the influence of the load P at A has exactly the same shape as the elastic curve produced along the line L_A under the influence of the load P at point B. In applying this conclusion to Figure 8 or Figure 9, let the line L_A be the edge shown on the drawing, and let the line L_B be at some distance from the edge. By the direct use of the diagrams one obtains the elastic curve at any line L_B parallel to the edge, due to a load at the edge. But one may interpret this curve as the elastic curve for the edge produced under the influence of a load at a point of the line L_B. The curvature of the deflected middle surface at point A of the edge in the direction of the edge, produced by the load P at any point B at some distance from the edge, is the same, accordingly, as the curvature of the deflected middle surface at point B in a direction parallel to the edge, as obtained in Figure 8 or Figure 9, due to the load P at the point A of the edge.

Thus Figures 8 and 9 may be used in studying the stresses produced along the edge by a wheel load at some distance from the edge.

The following use of the tables and diagrams is suggested. Let it be assumed that a certain pavement has been proved by tests and experience to be satisfactory for a given type of traffic. By the tables and diagrams one may compute, then, the corresponding critical stresses. These stresses may be adopted for the time being as allowable working stresses. With the stresses given, the tables and diagrams, through computations of the kind which has been shown, furnish answers to two questions: (1) What additional thicknesses are required if the wheel pressures are increased in a given manner; and, (2) what may be saved in the thicknesses by eliminating some of the heaviest vehicles. Prof. T. R. Agg has called attention to the importance of having an answer to the latter question, when one attempts to apportion the cost of the pavement to the various kinds of traffic for which it is used.

In using the tables and diagrams it should be kept in mind that the analysis is based on the assumptions which were stated at the beginning of this discussion. By the nature of these assumptions certain influences were left out of consideration, especially the following: (1) Variations of temperature, and other causes for tendency to change of volume; (2) the gradual diminishing of the thickness from the edge toward the interior; (3) local soft or hard spots in the subgrade; (4) horizontal components of the reactions of the subgrade; and (5) the dynamic effect, expressed in terms of the inertia of the pavement and subgrade. The horizontal components of the reactions of the subgrade, which are due to friction, may have a strengthening influence, especially at some distance from the edges, by causing a dome action in the pavement. As to the dynamic effects, with known values of the maximum pressure developed between the tire and the pavement, the effect of the inertia of the pavement may possibly be expressed approximately in terms of an increased value of the modulus, k. These additional influences are suitable subjects for further analysis.

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[9] The theory by which these diagrams were obtained may be found in a paper by the writer: "Om Beregning af Plader paa elastisk Underlag med særligt Henblik paa Spørgsmaalet om Spændinger i Betonveje," Ingeniøren (Copenhagen), v. 32, 1923, pp. 513-524. See also "Die elastischen Platten," by A. Nádai, (Berlin) 1925, p. 186.

TESTS OF VIBROLITHIC CONCRETE

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by **L. W. TELLER**, Engineer of Tests, and **C. E. PROUDLEY**, Assistant Engineer of Tests

THE Bureau of Public Roads, cooperating with the American Vibrolithic Corporation has recently completed 28-day tests on concrete slabs constructed of the same materials in accordance with ordinary methods and the Vibrolithic process patented by the company.

The tests were made for the purpose of obtaining data on the relative strength of specimens equivalent in every particular except the method of construction. To that end every effort was made to eliminate all variables except the methods of placing, tamping, and finishing, and to have all operations performed under as nearly similar working conditions as possible.

Although it is recognized that the ultimate problem is one of economy, the current investigations have been confined to the study of certain physical properties of slabs made by the two methods; among them, the tensile strength as determined by bending, density, coefficients of expansion, and uniformity of the product.

The program of tests includes, in addition to those which have been made at the end of 28 days, others that will be made after one year; and the formulation of definite conclusions must await the completion of the latter tests. There are certain indications, however, which may be noted at this time with the understanding that they may be modified by the data obtained from the remaining tests.

Giving due consideration to all features of the investigation, such as the workability of the concrete and the method of finishing the normal specimens the following tentative conclusions may be drawn from the bending tests at the age of 28 days.

1. The Vibrolithic process resulted in a more uniform product.
2. For a given cement content, the slabs constructed by the Vibrolithic method exhibited greater strength than the normal concrete.
3. The strength of the slabs constructed by the Vibrolithic process, when tested with tension in the bottom, was practically the same as when tested with tension in the top.
4. The strength of the normal concrete, when tested with tension in the bottom, was less than when tested with tension in the top.

DIMENSIONS AND CONSTRUCTION OF THE TEST SLABS

The 36 by 72 inch slabs of each class were constructed of the same kinds of materials under conditions as nearly uniform as practicable. Designed to be 6 inches in depth, the individual slabs actually varied slightly from this dimension as indicated in Tables 1 and 2.

The slabs were constructed on a specially prepared subgrade which was sprinkled and rammed thoroughly with a 20-pound tamper the day before placing the concrete and sprinkled again the morning the slabs were poured. The 2 by 6 inch oiled, dressed-lumber outside forms set on the subgrade thus prepared inclosed a row of five slabs which were formed by 2 by 4 inch separators, the beveled upper edges of which were set 2⅛ inches below the top of the outside forms, as shown in Figure 1.

The cement used was a brand of known reputation and predetermined satisfactory quality. All cement for a day's run was thoroughly mixed and resacked in lots of 47 pounds each and one of these sacks was tested each day in the laboratory with the results shown in Table 3 and Figure 12. Other physical properties of the cement were as follows:

Specific gravity	3. 10
Fineness, retained on 200-mesh sieve, per cent	15. 1
Soundness	O. K.
Time of set (Gilmore needle):	
Initial	3 hrs. 25 min.
Final	6 hrs. 55 min.
Normal consistency, per cent	23. 0

The aggregates used in all slabs—Potomac River sand and limestone in two sizes obtained from Frederick, Md.—had the following grading and physical properties.

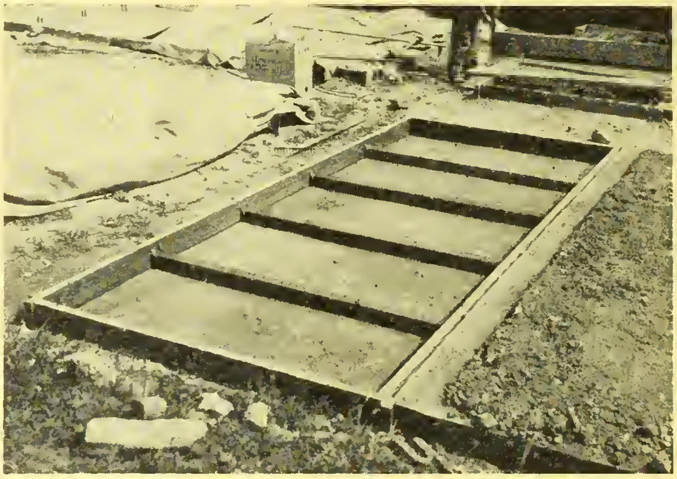

FIG. 1.—Forms and separators in place

GRADING AND PHYSICAL PROPERTIES OF THE SAND

Passing ¼-inch screen	per cent	91
10-mesh sieve	do	76
20-mesh sieve	do	65
30-mesh sieve	do	47
40-mesh sieve	do	35
50-mesh sieve	do	20
100-mesh sieve	do	7
200-mesh sieve	do	4
Loss by washing	do	2. 9
Tensile strength ratio:		
7 days	do	114
28 days	do	118
Weight per cubic foot, dry	pounds	105
Organic matter		immaterial coloration.

Bulking with 4 to 6 per cent moisture, approximately 20 per cent.

GRADING AND PHYSICAL PROPERTIES OF THE STONE

	Small stone per cent	Large stone per cent
Passing 2-inch screen		98. 6
1½-inch screen	100. 0	64. 7
1-inch screen	84. 4	6. 0
¾-inch screen	42. 9	1. 2
½-inch screen	12. 3	1. 2
¼-inch screen	1. 8	0
10-mesh sieve	1. 0	
Percentage of wear		4. 2
Hardness coefficient		17. 5
Toughness		17
Specific gravity		2. 70
Absorption, per cent		. 10
Weight per cubic foot, solid, pounds		168
Crushed:		
Small stone, do		95
Large stone, do		98
Mixed 50-50, do		100

In order to obtain a unit volume of the mixture in equal parts of large and small stone it was found that 12½ per cent additional volume of each was required; i. e., a mixture of 1⅛ cubic feet of each size yielded 2 cubic feet.

To compensate for the bulking of the sand and the oversize material in it, 1¼ cubic feet of damp sand was used volumetrically as the equivalent of 1 cubic foot of dry sand. The small quantity of coarse sand considered as of gravel size did not affect the volume of stone in the proportions used. Figure 2 shows the grading curves of the aggregates used.

One bag of cement weighing 94 pounds was assumed to be equal to 1 cubic foot; and cubic-foot boxes, carefully marked in quarters, were used for measuring the sand and stone. The volumes of materials actually used in each batch were as follows:

NORMAL CONCRETE

	Proportions			
	1 : 1½ : 3	1 : 2 : 3	1 : 2 : 3½	1 : 2 : 4
Cement, bags	1½	1½	1½	1
Sand (damp)________cubic feet	2¾	3¾	3¾	2½
Stone:				
Large________do	2½	2½	3	2¼
Small________do	2½	2½	3	2¼

VIBROLITHIC CONCRETE

	Proportions			
	1 : 1½ : 3½	1 : 2 : 3½	1 : 2 : 4	1 : 2 : 4½
Cement, bags	1½	1½	1	1
Sand (damp)________cubic feet	2¾	3¾	2½	2½
Stone:				
Large________do	3	3	2¼	2½
Small________do	3	3	2¼	2½

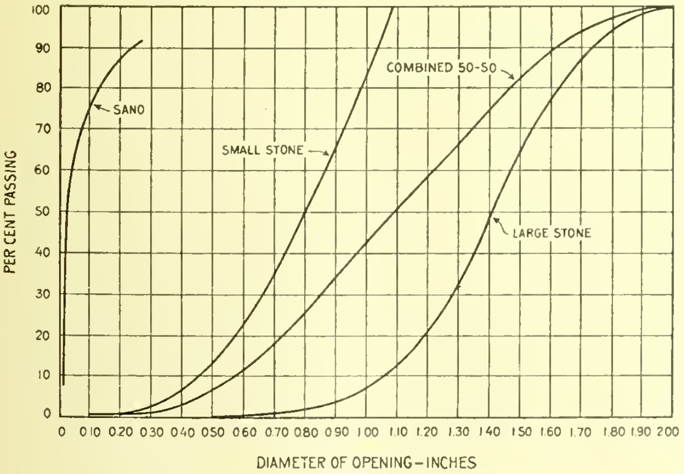

FIG. 2.—Grading of aggregates used in the concrete

The 10-cubic-foot, drum-type mixer, driven electrically at the rate of 20 r. p. m., was charged with the proper quantities of stone, sand, and cement in the order named, followed by sufficient clear Potomac River water to give the required consistency. The water tank was equipped with a gage glass calibrated to quarters of gallons. As the concrete was discharged from the mixer a flow determination was made on a portion of each batch, and as nearly as possible, the consistency was regulated so as to give a flow of 110 to 115 on the 30-inch flow table. As a further check on the consistency the second and sixth batches of every run were tested for slump with the approved slump cone, the requirement being a slump of about 2 inches.

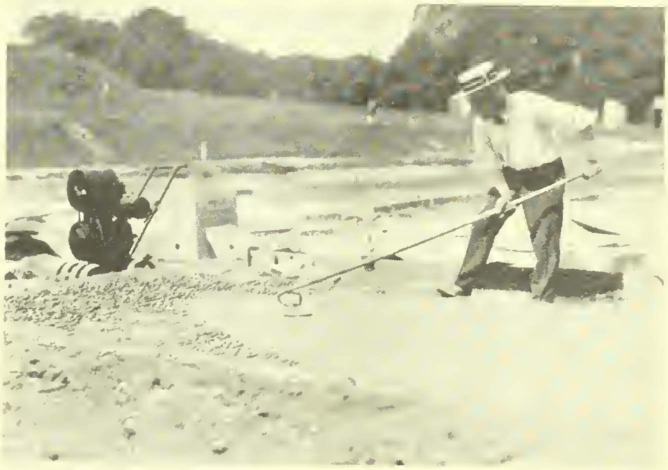

FIG. 3.—Finishing Vibrolithic surface with steel float prior to belting

The concrete, which was placed in the forms without regard for the separators, was tamped and struck off with a straight 2 by 12 inch strike board worked lengthwise over each series of five slabs, and the desired finish of the normal concrete was obtained by belting lengthwise over the slabs with an 8-inch rubber belt.

The Vibrolithic sections were similarly placed and struck off, after which they were covered uniformly with Frederick limestone of 2-inch to 1-inch size at the following rates:

Mix	1:1½:3½	1:2:3½	1:2:4	1:2:4½
Pounds per square yard	50	50	45	40

On this stone were placed special racks over which the vibrators were run, according to the patented process. Approximately four minutes of vibration was allowed for each slab. The removal of the racks left an irregular mortar surface which was smoothed down with a long-handled steel float as shown in Figure 3, after which the surface was belted. Dry spots were sprinkled with a little water to facilitate finishing.

All slabs, as soon as their hardness would permit, were covered with wet burlap which was kept damp during the day and thoroughly wet down before leaving at night. The following morning it was removed and replaced with a covering of damp earth. The earth was kept damp by daily sprinkling until 28 days had elapsed, at which time the top surface was cleaned. After the removal of the earth the slabs were kept damp by means of wet burlap until time for testing. Slabs to be tested at one year will remain uncovered 10 months, but will again be covered with damp earth for the 28 days immediately prior to testing.

TABLE 1.—*Summary of data of tests on normal concrete slabs*

Mix	Slab No.	Surface in tension	Depth	Section modulus, $\frac{I}{c}$	Total load at rupture	Modulus of rupture			Strength ratio = tension in top / tension in bottom	Variation of individual tests from average of groups	
						Individual slabs	Averages				
			Inches		*Pounds*	*Pounds*	*Pounds*			*Per cent*	
1:1½:3	86	Top	6.1	227.2	17,810	784				12.0	
	88	do	6.2	238.0	18,010	757				8.2	
	90	do	6.3	241.3	15,610	647	700			7.6	8,1
	97	do	6.4	253.1	17,660	698				.3	
	99	do	6.5	260.7	16,010	614				12.2	
	87	Bottom	6.0	218.4	13,960	[1] 551		624	1.275	.4	
	89	do	6.2	233.6	12,410	532				3.1	
	96	do	6.3	239.7	15,510	647	549			17.8	7,4
	98	do	6.4	248.3	12,560	506				7.8	
	100	do	6.5	255.0	12,910	507				7.7	
1:2:3	116	Top	6.1	227.2	14,460	636				5.0	
	118	do	6.2	238.0	13,660	574				5.3	
	120	do	6.1	227.2	14,310	630	606			4.0	5.7
	167	do	6.2	238.0	15,210	639				5.5	
	169	do	6.2	238.0	13,160	553				8.8	
	117	Bottom	6.1	225.5	13,160	584		595	1.039	.2	
	119	do	6.0	218.4	10,960	502				13.9	
	166	do	6.4	247.3	16,680	675	583			15.8	6.7
	168	do	6.2	233.6	13,220	566				2.9	
	170	do	6.3	239.7	14,080	588				.9	
1:2:3½	127	Top	6.3	245.4	14,360	585				2.5	
	129	do	6.2	238.0	14,840	623				3.7	
	136	do	6.5	256.6	15,560	607	600			1.2	5.2
	138	do	6.5	260.7	16,920	649				8.1	
	139	do	6.7	277.3	14,910	538				10.3	
	126	Bottom	6.4	247.3	13,760	557		586	1.047	2.8	
	128	do	6.3	240.7	14,210	590				3.0	
	130	do	6.5	255.0	14,860	583	573			1.7	2.8
	137	do	6.5	255.9	13,980	546				4.7	
	140	do	6.6	262.1	15,260	582				1.6	
1:2:4	147	Top	6.0	224.3	11,360	507				9.5	
	149	do	5.9	216.4	10,510	486				13.2	
	156	do	6.1	227.2	14,340	631	560			12.7	10.1
	158	do	6.2	238.0	15,000	630				12.5	
	160	do	6.6	265.3	14,460	545				2.7	
	146	Bottom	6.2	232.2	9,760	420		507	1.233	7.5	
	148	do	5.9	212.0	8,920	421				7.3	
	150	do	6.2	232.2	9,860	424	454			6.6	8.7
	157	do	6.3	240.7	12,720	529				16.6	
	159	do	6.4	248.3	11,860	478				5.3	
Mean									1.149	Top 7.5 Bottom 6.4	

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[1] Broke 17¼ inches from end support.

TABLE 2.—*Summary of data of tests on Vibrolithic concrete slabs*

Mix	Slab No.	Surface in tension	Depth	Section modulus, $\frac{I}{c}$	Total load at rupture	Modulus or rupture			Strength ratio = tension in top / tension in bottom	Variation of individual tests from average of groups	
						Individual slabs	Averages				
			Inches		*Pounds*	*Pounds*	*Pounds*			*Per cent*	
1:1½:3½	82	Top	6.1	231.5	16,536	714				0.1	
	84	do	6.2	238.0	16,286	684				4.3	
	91	do	6.1	227.2	16,386	721	715			0.9	4.1
	93	do	6.2	238.0	16,036	674				5.7	
	95	do	6.1	227.2	17,786	783				9.6	
	81	Bottom	6.1	224.2	16,486	736		708	1.020	5.0	
	83	do	6.2	233.6	17,086	732				4.5	
	85	do	6.1	224.2	14,036	626	701			12.1	4.8
	92	do	6.0	218.4	15,586	714				1.8	
	94	do	6.1	225.5	15,686	696				0.7	
1:2:3½	112	Top	5.9	216.4	14,336	662				3.4	
	114	do	6.1	231.5	13,986	604				11.8	
	161	do	6.2	233.8	18,106	774	685			13.0	7.1
	163	do	6.1	231.5	16,586	717				4.7	
	165	do	6.2	233.8	15,566	666				2.8	
	111	Bottom	6.3	239.7	16,886	705		669	1.047	7.8	
	113	do	6.2	233.6	15,136	648				0.9	
	115	do	6.4	247.3	13,336	539	654			17.6	7.4
	162	do	6.4	248.3	16,636	670				2.5	
	164	do	6.3	240.7	16,986	706				8.0	
1:2:4	121	Top	6.4	248.9	15,006	603				2.6	
	123	do	6.3	245.4	14,286	582				1.0	
	125	do	6.5	256.6	15,136	590	588			0.4	1.9
	132	do	6.2	238.0	14,236	598				1.7	
	134	do	6.1	231.5	13,126	567				3.6	
	122	Bottom	6.3	240.7	13,026	542		579	1.030	5.1	
	124	do	6.2	233.6	13,586	582				2.0	
	131	do	6.5	255.0	14,406	565	571			1.1	4.1
	133	do	6.3	240.7	14,866	617				8.1	
	135	do	6.2	232.2	12,726	548				4.0	
1:2:4½	141	Top	6.3	241.3	12,986	538				5.9	
	143	do	6.0	224.3	12,646	564				1.4	
	145	do	6.1	227.2	13,986	615	572			7.5	5.6
	152	do	6.3	245.4	13,086	533				6.8	
	154	do	6.3	245.4	14,960	610				6.6	
	142	Bottom	6.0	218.4	13,306	610		578	0.981	4.7	
	144	do	6.2	233.6	11,586	496				14.9	
	151	do	6.3	239.7	15,586	651	583			11.7	7.4
	153	do	6.3	240.7	13,566	563				3.4	
	155	do	6.6	262.1	15,646	597				2 4	
Mean									1.020	Top 4.7 Bottom 5.9	

THE METHOD OF TESTING

When the slabs were ready to be tested they were lifted from the subgrade by means of a cradle of the special design shown in Figure 4. As, in nearly every case, cracks had formed over the separators there was no difficulty in raising the individual slabs; but in most instances a large amount of subgrade came up with the slabs and had to be removed.

Half of the slabs were tested with tension in the top surface, the others with tension in the bottom, the former condition being effected by inverting the slab in the testing machine. In order to obtain a solid bearing for the knife edges on the under surface of the slabs, flat steel strips 1¼ inches wide and three-sixteenths inch thick were set in plaster of Paris in the proper position. These strips were not needed on the finished surface, as the rubber pads on the knife edges, shown in Figures 5 and 6, took up the slight irregularities which, in general, were merely the marks of the float or belt and were not more than one-sixteenth inch deep.

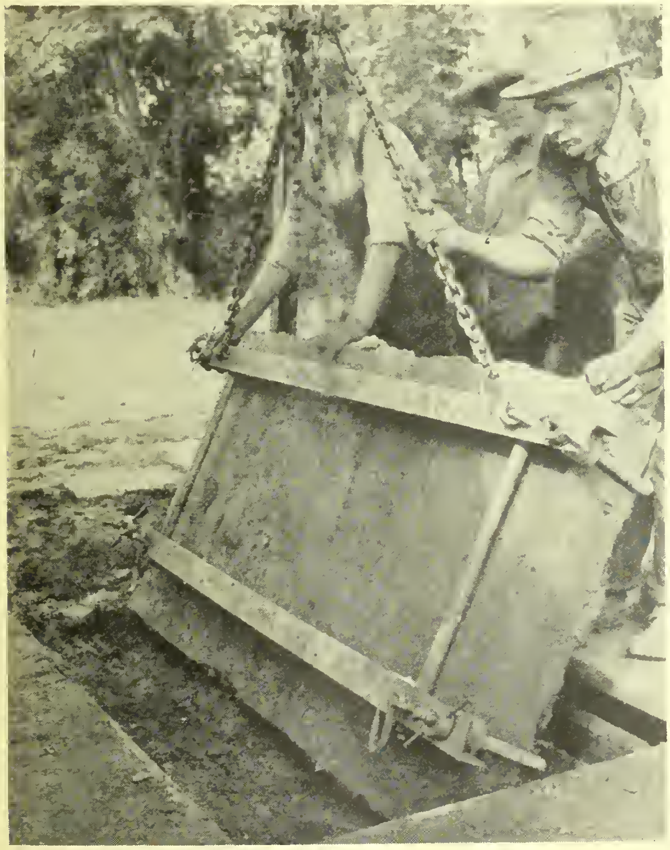


Fig. 4.—Lifting slab from subgrade

Referring to Figure 7, which shows the elevation of the testing machine, the method of testing may be described as follows: The slab, *A*, was carefully centered on the lower knife edge, *B*, and the rocking knife edge, *C* (see also fig. 5). The upper knife edges, *D*, mounted with the hydraulic jack, *E*, and the calibrated head, *K*, on the carrier plate, *G*, was swung into position over the slab so that loading was at the third points of the slab. The collar, *I*, kept the carrier plate elevated so that the knife edges would swing clear of the slab until in position, at which time the screw jack, *J*, was used to lower guide post, *H*, to which the collar was fastened. The collar was lowered until the carrier plate was entirely free. The follower head, *L*, carried by the plate, *M*, was then swung about guide post, *H*, and brought into contact with the top of the machine by means of hand screw, *N*. The initial load was then put on the slab by means of the jack handle of hydraulic jack, *E*, after which the load was continuously and steadily applied by means of the small auxiliary pump, *F*. The calibrated beams, Figure 8, were arranged to deflect as simple beams as the load was applied. The dial, read initially before the load was applied, was watched carefully by at least two observers so as to catch the maximum dial reading, and a calibration chart was used for converting the difference in dial readings to total applied load in pounds.

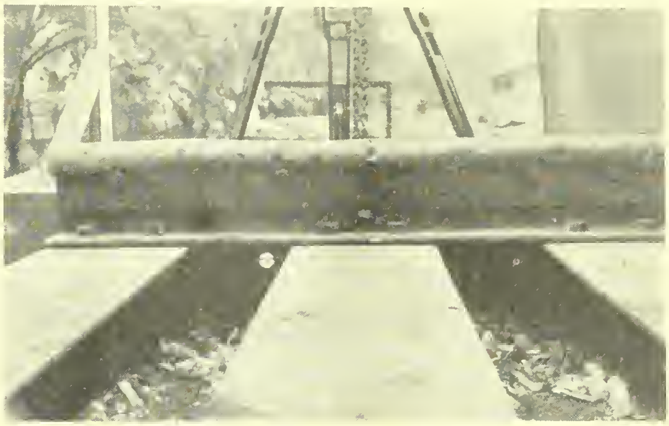


Fig. 5.—Lower knife edge showing rocker pin and rubber pad

The broken sections were removed and stacked with their broken faces outward, as shown in Figures 10 and 11, and their cross sections were then measured by two operators each of whom made four depth measurements on each slab.

COMPUTATION OF MODULUS OF RUPTURE

The weight of the slab itself, the weight of the upper knife edges and loading device, and the pressure applied by the jack comprised the total load. The uniformly distributed weight of the slab was converted into a concentrated load which could be added to the weight on the knife edges. Thus, the weight of the 1:2½:3 normal concrete slabs, 152 pounds per cubic foot, was found to be equivalent to two loads of 410 pounds each concentrated at the knife edges; and the 1:1½:3½ vibrolithic slabs which weighed 157 pounds per cubic foot were equivalent to two concentrated loads of 424 pounds. The dead load of the knife edges, jack, etc., was found to be 640 pounds, or at each knife edge, 320 pounds.

From an initial load computed in this manner the pressure on the slabs was gradually increased by means of the hydraulic jack, *E* (fig. 7), until rupture occurred, at which time the pressure indicated by the calibrated beams added to the initial dead load constituted the total load, *P*.

The extreme fiber tensile stress, S, was then computed from the formula

$$S=M\frac{c}{I}$$

in which the bending moment for the 60-inch span with third-point loading is

$$M=\frac{P}{2}\times 20=10P.$$

The section modulus, $\frac{I}{c}$, computed from the measurements of the broken sections, was found to be different in almost every specimen, principally because of variations in the thickness of the slabs and in the location and number of the separator projections on the edges. The projections, or sections of the slabs formed over the intermediate separating forms, were trapezoidal in cross section, approximately seven eighths inch in width and from 2⅛ to 3 inches in depth.

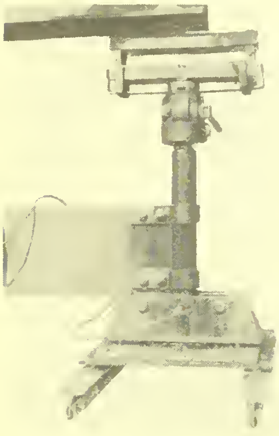


Fig. 6.—Arrangement of loading knife edges

The thicknesses of the several slabs, their section moduli, the total loads at rupture, and the computed moduli of rupture are shown for the two classes of concrete in Tables 1 and 2, respectively. The moduli of rupture are shown also in Figure 12 and Table 3, with other factors indicative of the characteristics of the concrete.

These other factors include the tensile strength of 1:3 mortar briquettes made of the cement used in the slabs and Ottawa sand; the modulus of rupture of 1:2 Ottawa sand mortar beams; the consistency of the concrete as indicated by its flow; the water-cement ratio; and the ratio of the cement to aggregate by weight for each of the slabs.

The water-cement ratio is a volumetric relation and assumes 1 bag of cement to equal 1 cubic foot. Thus, if a batch of concrete were mixed with 7½ gallons of water for each bag of cement, the water-cement ratio, $\frac{W}{c}$, would be recorded as 1.00. In computing the relation the quantity of moisture contained in the sand, as found by a daily moisture determination, was added to the volume of water introduced in mixing to obtain the total value of W.

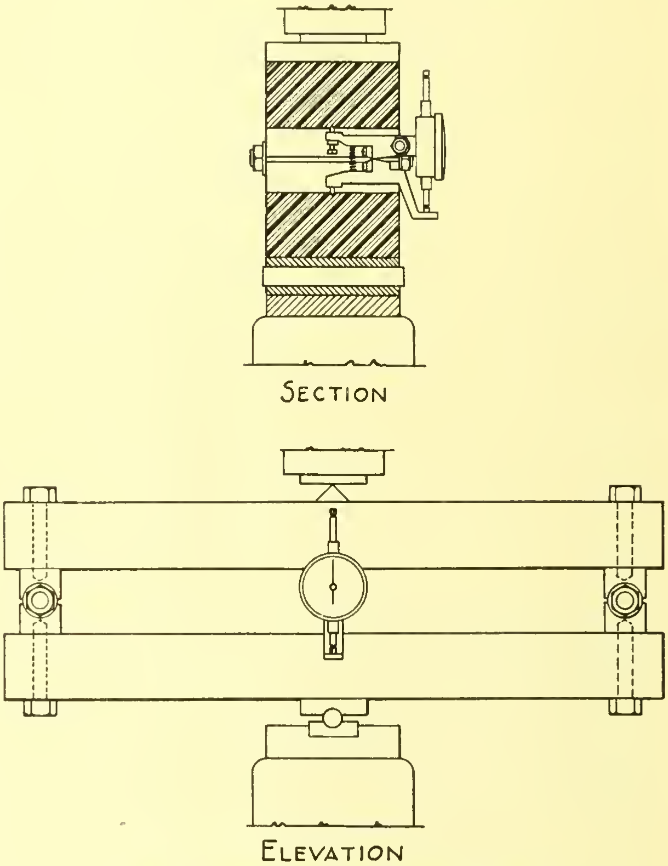


Fig. 8.—Calibrated beams used to measure the load

The cement-aggregate ratio is based on the nominal mix for both Vibrolithic and normal concrete. The top stone of the Vibrolithic concrete is not included

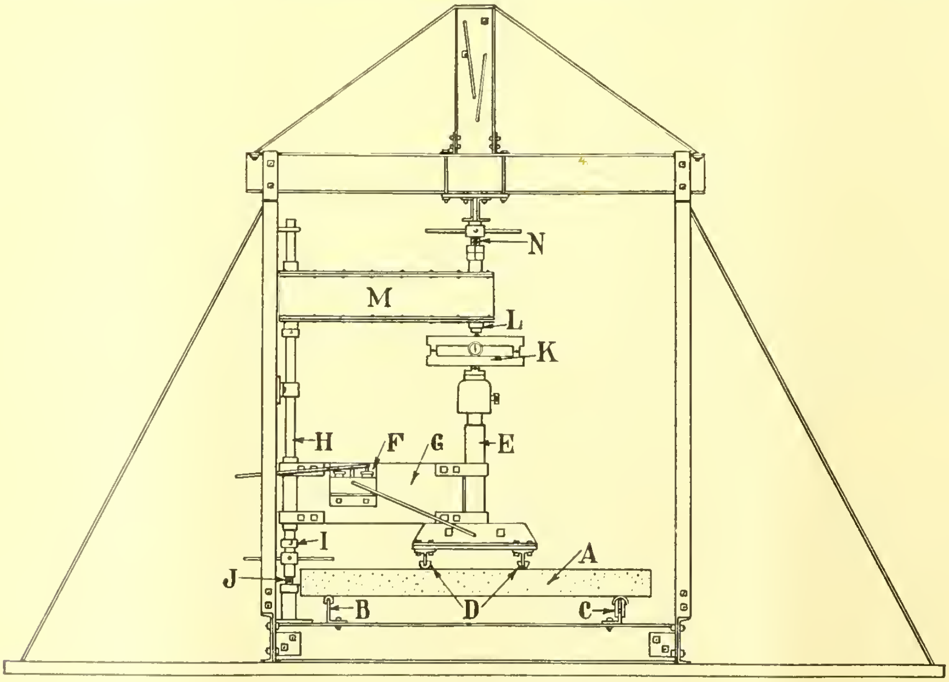


Fig. 7.—Apparatus for testing the slabs

with the aggregate in computing the relation. Using the weights per cubic foot of the sand and stone as determined by tests, the volumetric proportions were readily converted to weights; and the weight of cement divided by the total weight of sand and stone gives the ratio.

CEMENT-AGGREGATE AND WATER-CEMENT RATIOS AS STRENGTH INDICES

Past experience naturally leads to the expectation that the strength of concrete will follow, in general, the cement content. Table 3 and Figures 9 and 12 show that the expected relationship obtained in these tests, the cement content being expressed as the ratio of cement to total aggregate in the mix.

The water-cement theory is also seen to agree with the strengths of the slabs. The leaner mixes were gauged with higher water-cement ratios and consequently gave lower strength. It is seen, however, that there is practically no difference between the average strength of the 1:2:3 and the 1:2:3½ normal concrete, although the other data do not show the reason for this.

Similarly, there is no difference between the average strength of the 1:2:4 and the 1:2:4½ Vibrolithic concrete. In this case, however, the $\frac{W}{c}$ is the same for each of these proportions. This probably accounts, in part at least, for the uniformity in strength. It is also interesting to note in this connection that the quality of the cement, as indicated by the tension tests of mortar, was lower for that used in the 1:2:4 mix than in the 1:2:4½ mix, whereas the quality as indicated by the cross-bending test of 1:2 Ottawa sand mortar beams is the reverse. It is probable that a more thorough examination of the slabs as called for by the program of tests will furnish information that will explain some of these deviations.

The relation between the quantity of cement used and the strength obtained is shown in Figure 9. For equivalent quantities of cement the strength of the Vibrolithic concrete is higher than the normal concrete. The difference amounts to about 16 per cent for 1:2:3 and 1:1½:3½ concrete, and about 21 per cent for 1:2:4 concrete based on the strength of the normal concrete.

Considering the two processes from the standpoint of the cement required to give equal strength, a comparison at 600 pounds per square inch modulus of rupture shows that roughly, 19 per cent additional cement is required in normal concrete; or, in other words, only 84 per cent of the cement required for normal concrete is necessary for equal strength in Vibrolithic work. At higher strengths the curves (fig. 9) indicate slightly greater advantages for the Vibrolithic, and at lower strengths less advantage insofar as saving in cement is concerned.

TOP AND BOTTOM STRENGTH OF VIBROLITHIC SLABS NEARLY EQUAL

Due to difference in density and, possibly, to other variables as yet undetermined, the strength obtained when the bottom of the slab was in tension was, in most cases, lower than when the top was tested in tension. The average amount of this difference is shown graphically in Figure 12, the shaded bars giving the strength with the top in tension and the solid bars the strength with the bottom in tension. This is also shown in Tables 1 and 2.

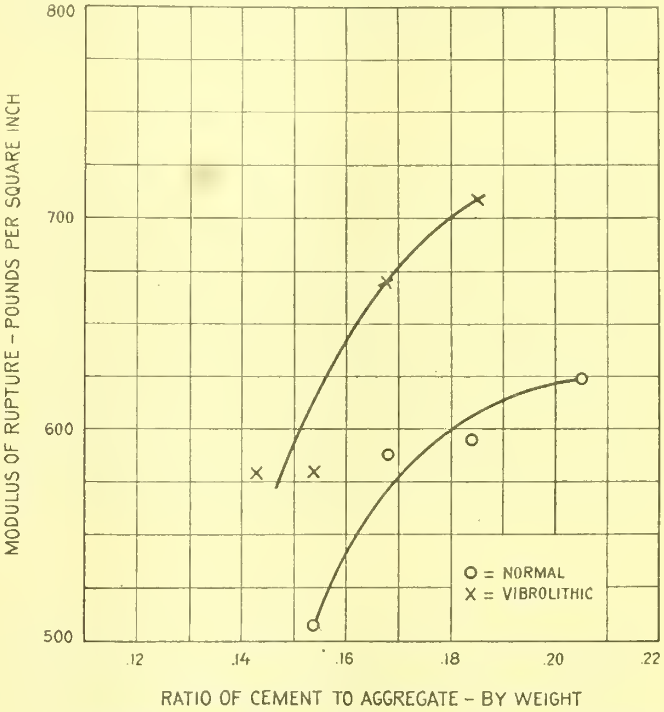
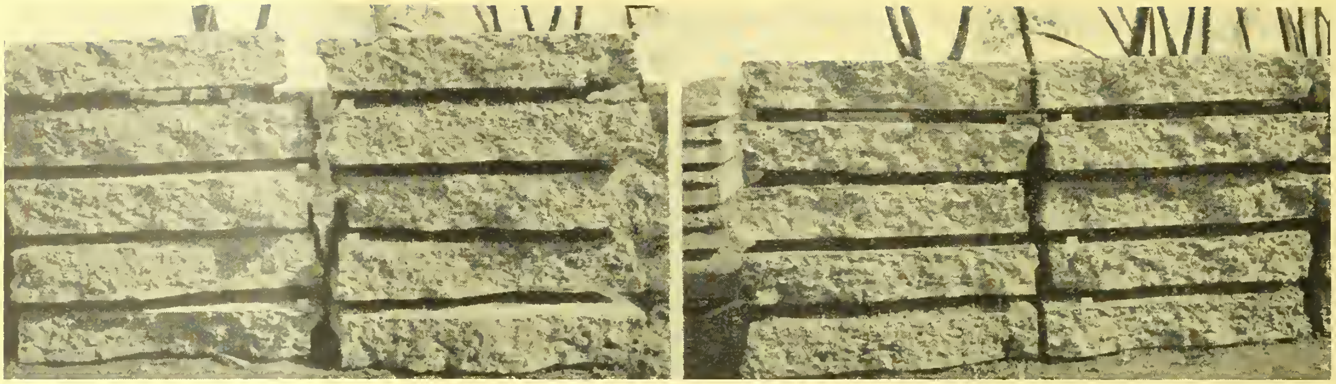


FIG. 9.—Relation of modulus of rupture of normal and Vibrolithic concrete to cement-aggregate ratio

It has been suggested that, as a result of excessive tamping and finishing, the strength of the upper surface of a concrete road might be increased beyond that of the bottom. These tests do not substantiate this theory. There is no doubt that the Vibrolithic method is the more vigorous finishing treatment; nevertheless, insofar as the uniformity of strength in top and bottom of the slab is concerned, the Vibrolithic is more remarkable than the normal concrete. Expressed numerically, the average resistance to tension in the bottom of normal concrete slabs is 87.7 per cent of that in the top, and for Vibrolithic it is 98.0 per cent.

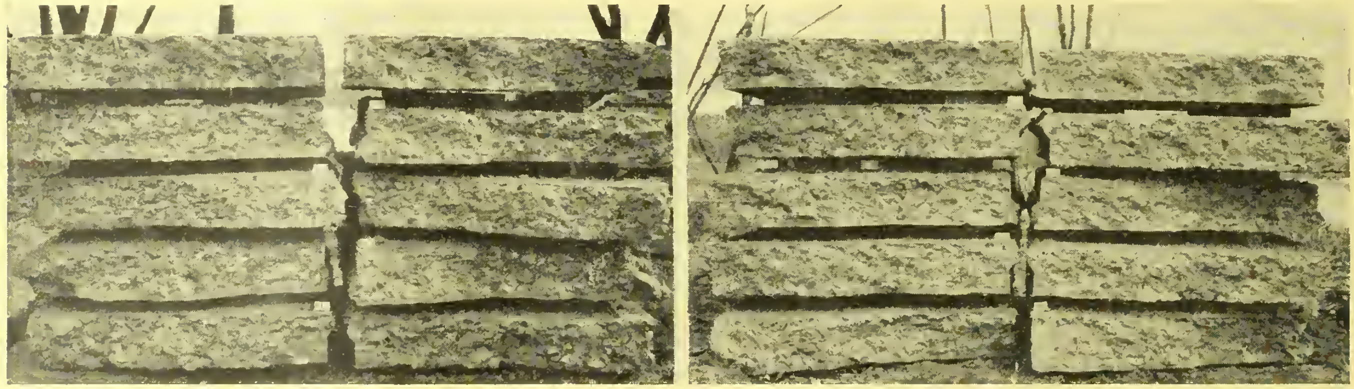
TABLE 3.—*Results of tests of slabs, aggregate, and cement*

Finish	Normal concrete								Vibrolithic concrete							
Mix	1:1½:3		1:2:3		1:2:3½		1:2:4		1:1½:3½		1:2:3½		1:2:4		1:2:4½	
Slab numbers, inclusive	86 to 90	96 to 100	116 to 120	166 to 170	126 to 130	136 to 140	146 to 150	156 to 160	81 to 85	91 to 95	111 to 115	161 to 165	121 to 125	131 to 135	141 to 145	151 to 155
Modulus of rupture concrete:																
Tension in top	729	656	613	596	604	598	496	602	699	726	633	719	592	582	572	571
Tension in bottom	542	553	543	610	577	564	422	503	698	705	631	688	562	577	553	604
Ratio of cement to aggregate by weight	0.205		0.184		0.168		0.154		0.185		0.168		0.154		0.143	
Water-cement ratio	0.67	0.73	0.78	0.77	0.88	0.89	0.86	0.86	0.69	0.75	0.80	0.80	0.91	0.91	0.91	0.90
Consistency flow table	112	112	114	112	111	112	109	108	110	113	110	109	107	107	114	112
Modulus of rupture 1:2 mortar	775	811	815	707	775	748	655	753	775	811	815	707	775	748	655	753
Tensile strength 1:3 mortar briquettes	385	335	355	355	345	380	385	395	385	335	355	355	345	380	385	395



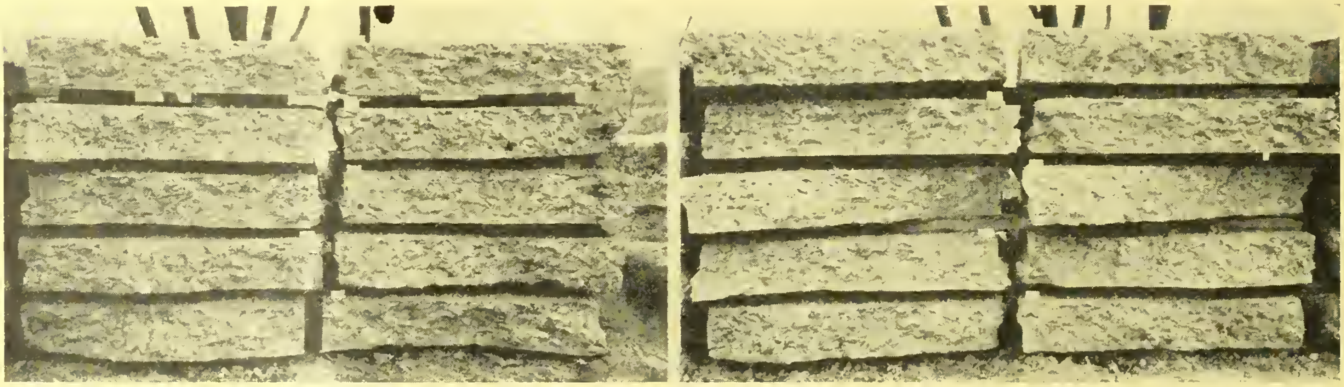
1:1½:3½ mix

1:1½:3½ mix



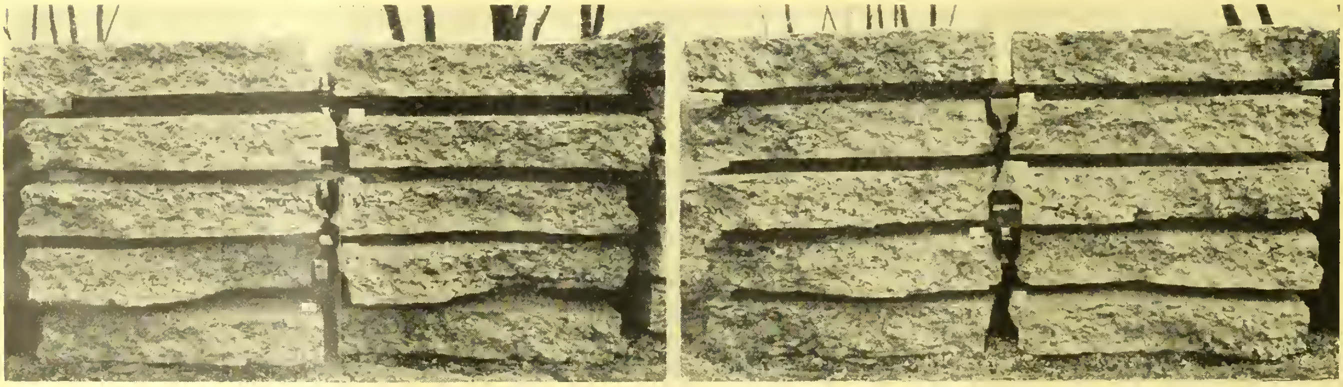
1:2:3½ mix

1:2:3½ mix



1:2:4 mix

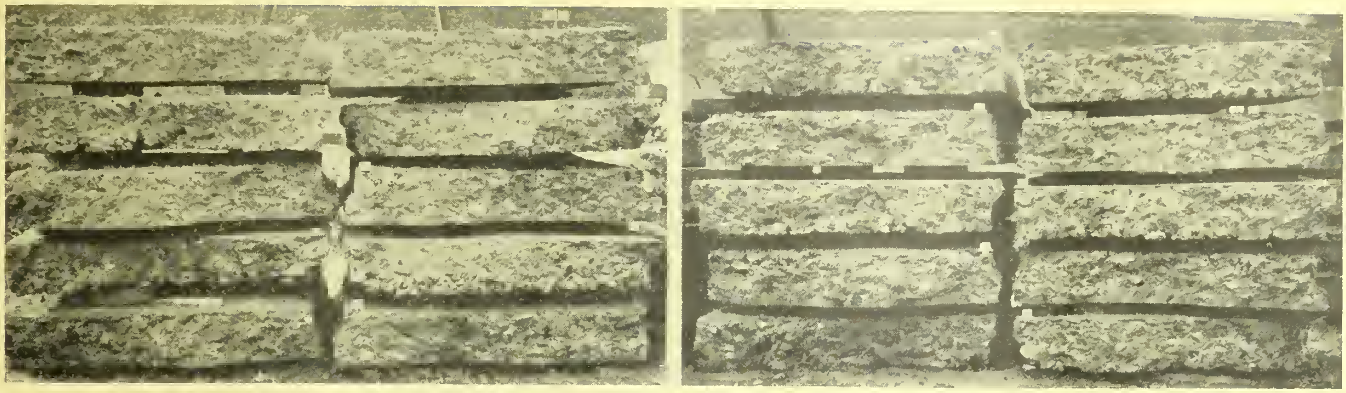
1:2:4 mix



1:2:4½ mix

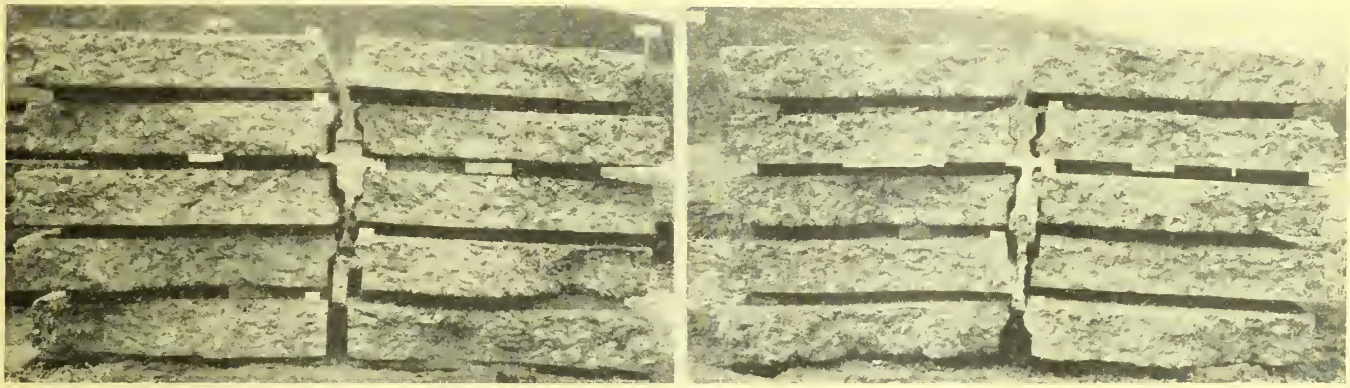
1:2:4½ mix

FIG. 10.--Vibrolithic concrete slabs of various mixes showing cross sections at point of failure in test



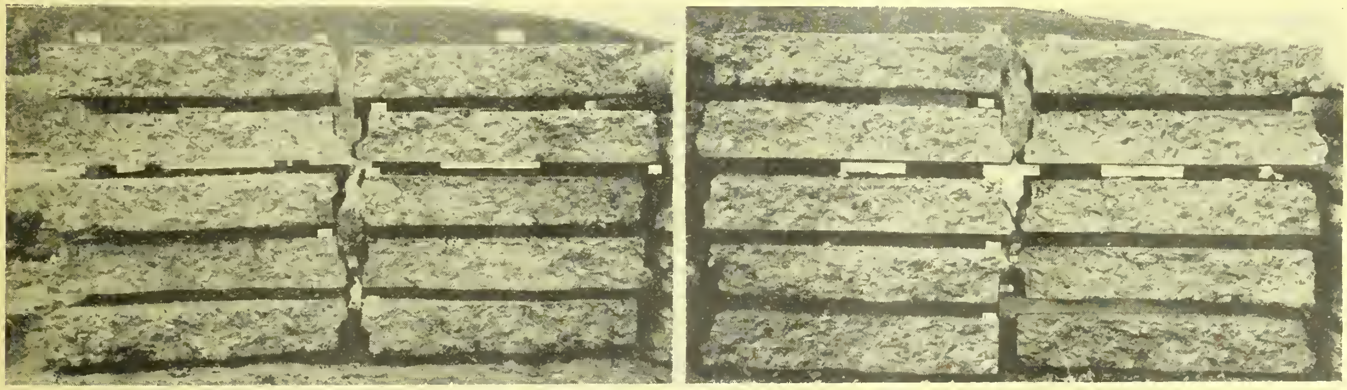
1: 1½ : 3 mix

1: 1½: 3 mix



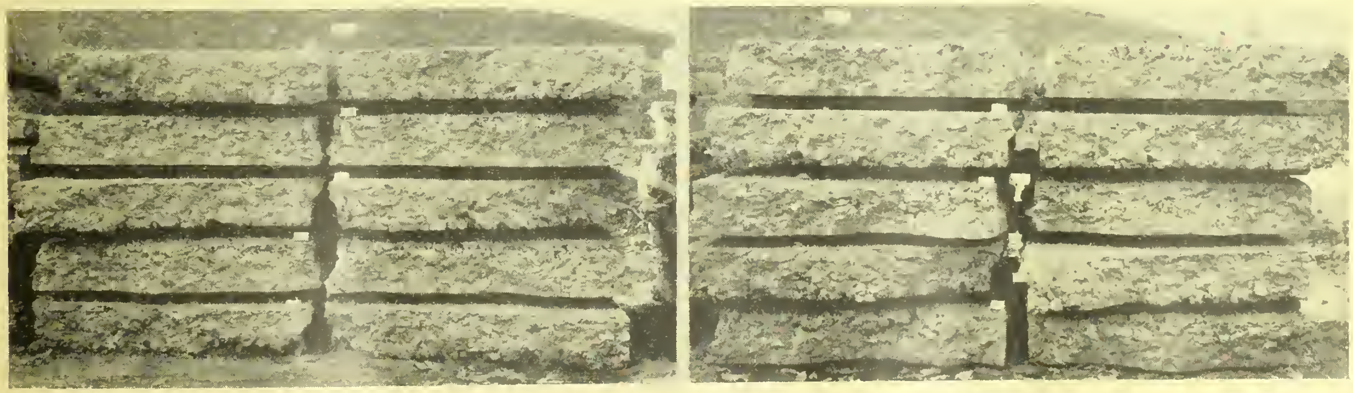
1: 2 : 3 mix

1: 2 : 3 mix



1: 2 : 3½ mix

1: 2 : 3½ mix



1: 2 : 4 mix

1: 2 : 4 mix

FIG. 11.—Normal concrete slabs of various mixes showing cross sections at point of failure in test

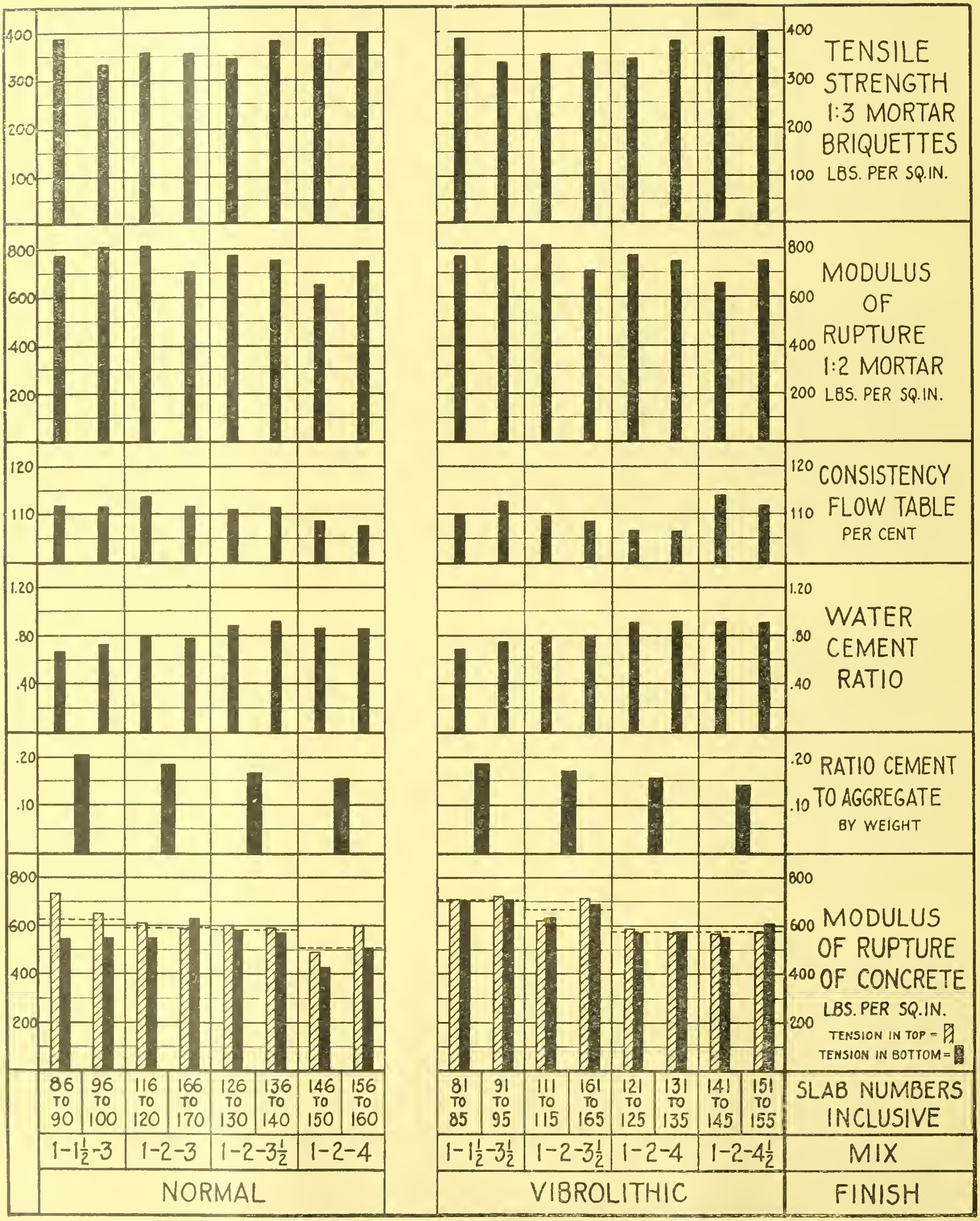


Fig. 12.—Results of tests of normal and Vibrolithic concrete and mortar briquettes and beams

Another important consideration not to be overlooked is that of uniformity of the product. It may be seen in the last column of Tables 1 and 2 that the average variation of the individual specimens from the average for the five specimens of the group is a little less in the case of the Vibrolithic than in the normal concrete. The average of all of these variations for Vibrolithic specimens is 5.3 per cent, and for the normal concrete specimens 6.8 per cent.

The photographs, shown in Figures 10 and 11, furnish an explanation of some of the apparent abnormalities in the results given in Tables 1 and 2. It is reasonable to suppose that, other things being equal, the slab with the minimum void spaces will give greatest strength; also, that the closer voids are to the outer surfaces, the more serious will be the effect on the strength. The distribution of void spaces in the specimens under consideration may be seen by a careful inspection of the photographs.

The 1:1½:3½ Vibrolithic specimens, Figure 10, show very few voids. Slab 94, which contains more than any other of this mix and type, shows one group of voids near the bottom and another a little below the center; but their effect is apparently negligible.

The 1:2:3½ specimens of Vibrolithic concrete are also quite dense except for a thin layer of voids occurring in slab 162 about one inch from the bottom, which seems to have had no serious effect on the strength.

The specimens of 1:2:4 Vibrolithic concrete exhibit a greater percentage of air pockets due, no doubt, to the leaner mix. It is interesting to observe the position of the voids in slabs 124, 131, 135, and especially in 121 and 122, where they are quite close to the neutral axis.

The 1:2:4½ mix of the Vibrolithic series apparently incloses a still greater number of voids, particularly in slabs 142, 143, 144, 151, 153, and 155. In 142 and 143 they are principally in the compression side as tested and probably have little effect except as they tend to lower the neutral axis and thus shorten the distance to the extreme fiber. In slab 144 a bad condition exists near the bottom, which is reflected in the unusually low strength obtained. The same is also true of slab 153. Slab 155 shows considerable void space, at an average distance of about 1½ inches from the bottom.

The richest mix in the normal concrete slabs, shown in Figure 11, is 1:1½:3. The most noticeable voids in slabs of this mix occur in Nos. 87, 88, 89, 97, 98, and 100. Of these slabs, 88 and 97 were effected only in the compression side, but the other specimens mentioned are penetrated varying distances up to 2½ inches in the tension side. These conditions, no doubt, explain some of the unusually low strengths obtained in this group.

The next leaner mix, 1:2:3, also shows many voids extending from the bottom well into the slab. Slabs 116, 120, 168, and 170 are badly honeycombed in the tension side, and in 117 and 119 the voids extend so deeply into the compression side as to cause a decrease in effective depth and consequent decreased strength when based on the slab thickness as measured.

In the 1:2:3½ mix of normal concrete, slabs 126, 130, 137, and 140 are most notable for their honeycombing, although every slab of this group includes an appreciable quantity of voids. The 1:2:4 normal concrete is in particularly poor condition due, very likely, to the leanness and dryness of the mix. The consistency as measured by the flow table was no less workable, however, than much of the concrete used in present-day construction of first-class concrete pavement on which a machine finishing is used.

EFFECT OF FINISHING METHODS OF NORMAL SLABS

Special consideration should be given the method of constructing the normal concrete slabs for this investigation, as it is plainly upon this factor that the value of these tests depends.

The concrete for the normal specimens was dumped into the forms, shoveled into place, and spaded along the edges of the forms. The specimens were tamped across once with the 2 by 12 inch strike board, after which the surface was struck off and belted to a finish.

As the consistency was held to the same value used for the Vibrolithic specimens, it was more nearly that which should be used for machine finishing, and the hand finishing described was not adequate. This was not apparent until the specimens were turned over at the time of testing, but the result was an unfortunate amount of honeycombing and consequent variation in the test results.

It is believed that, if the concrete in the normal specimens had been more thoroughly compacted as would be the case in machine finishing, less variation in strength would have resulted and generally higher values would have been obtained. This is indicated by the fact that those specimens where the lack of compaction was most apparent showed noticeably lower resistance to cross-bending.

An interesting general observation regarding the location of the void spaces in the specimens thus far tested is to be made from an inspection of the photographs and much better, of course, by a study of the actual specimens. Through the action of the vibrator on the surface of the concrete in the Vibrolithic process, mortar is apparently worked down to the subgrade, thus leaving the air spaces at some distance from the bottom of the slab. When the ordinary methods of finishing are used the voids which occur at the bottom while the concrete is being deposited remain unfilled, as a result of the arch action of the coarse aggregate. Advantages of concrete made by the Vibrolithic method, as brought out in this report, may be traced in many instances to the more favorable location of voids.

TEMPERATURE AS A FACTOR IN THE STABILITY OF ASPHALTIC PAVEMENTS

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by W. J. EMMONS, Highway Research Specialist, and B. A. ANDERTON, Chemical Engineer

CERTAIN asphaltic paving mixtures, notably those containing excessively high percentages of bitumen for the accompanying aggregates, are susceptible to displacement under traffic. Such weakness is generally manifested during the warmer seasons of the year when the bitumen, present in space-filling volumes of considerable magnitude, absorbs sufficient radiant energy from the sun to render it fluid. The pavement then exists as a structure composed of a more or less independently stable aggregate and a viscous fluid of low binding power. When, under these conditions, the pavement is subjected to traffic of sufficient magnitude the resisting power of the plastic mass is soon overcome and the development of ruts and waves occurs.

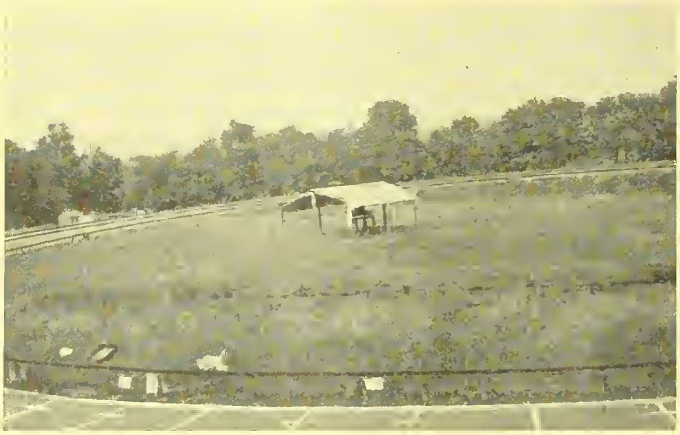


FIG. 1.—General view of experimental track surfaced with asphaltic pavements showing central temperature measurement station and thermo-couple wires

During the past three years the Bureau of Public Roads has conducted a series of tests to measure the internal temperatures of asphaltic pavements and to determine in a general way the effect of such temperatures upon the resistance to displacement under traffic. These tests were conducted in connection with two series of stability experiments which involved the construction of 60 sections of pavement on a circular track or roadway 13 feet in width. The mixtures included coarse-graded asphaltic concretes and sheet asphalts and were laid 2 inches thick upon a very smooth concrete foundation.

For the purpose of determining their relative displacement under traffic, screws were driven into the pavement across the line of traffic and referenced to permanent markers set in the concrete. Thermocouples of copper and constantin wire were installed in many of the pavements and connected to a central station for the observation of temperatures. Most of the thermocouples were set in the pavements at a depth of one-half inch below the surface, although in several instances additional ones were placed at half-inch intervals down to the concrete foundation. Figure 1 shows the general location of the experimental pavements, the central temperature measurement station and the thermocouple wires leading to it. Observations of temperatures were made at 10 a. m. and 2 p. m. each day when the track was subjected to traffic. It was found that an average of the air temperatures at these times very closely approximated the average air temperature for the daily period of operation of the test. Moreover, the maximum air temperature during the day was recorded at approximately 2 p. m.

The first series of stability tests was made on 27 different coarse-graded asphaltic concrete mixtures. Construction was completed late in the year and the operation of a loaded 3-ton truck over the test sections was begun in October. During the autumn, winter, and early spring months the average air temperature at 2 p. m. was below 65° F. and all of the pavement mixtures remained rigid. Late in the following May a sudden and decided rise in temperature took place, maintaining for the following four weeks a 2 o'clock average of about 80° F.

The effect of this increased temperature was immediately evident. Although many mixtures remained entirely stable, a number began to shove and rut under the continuing traffic. This series of mixtures was subjected to 50,000 passages of the truck, of which 60 per cent was imposed during the period from October to May, without appreciable effect upon the contour of the pavement. The remaining 40 per cent during the season of high temperatures served virtually to destroy several of the weaker sections.

DISPLACEMENT GREAT DURING SUMMER SEASON

Twenty-eight sheet asphalt and five asphaltic concrete mixtures were tested in the second series. The sheet asphalts were laid directly upon the smooth

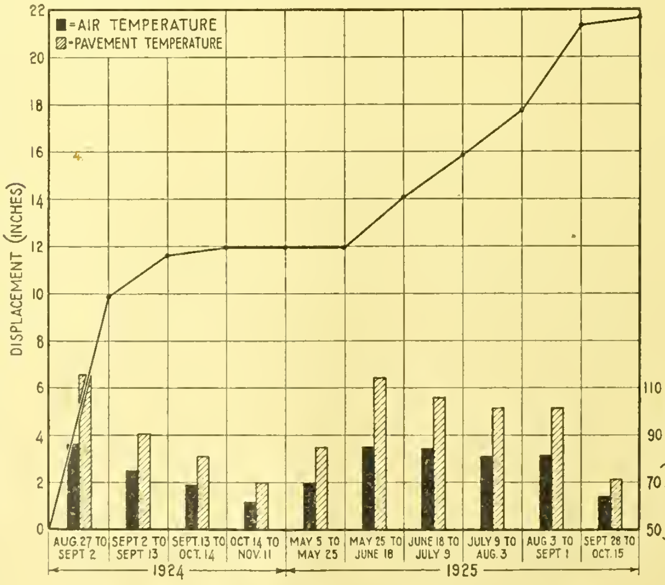


FIG. 2—Displacement and temperature of experimental asphaltic pavements

concrete foundation without the customary intermediate binder course. In this test, traffic was begun during the last few days of August and differences in the stability of the several mixtures were immediately developed. As the prevailing temperatures decreased the effect of traffic became less and less marked until, during the month preceding November 11, virtually no displacement was apparent. The average daily air temperature for the last period was 66°F. Traffic was suspended during the winter months, but was resumed early in the following May and continued until October 15. During this interval a complete cycle of temperature influences was observed. Stability of all mixtures at the low spring temperatures, shoving and rutting of weaker mixtures during the summer months, and once more an increased rigidity as cooler weather began to prevail.

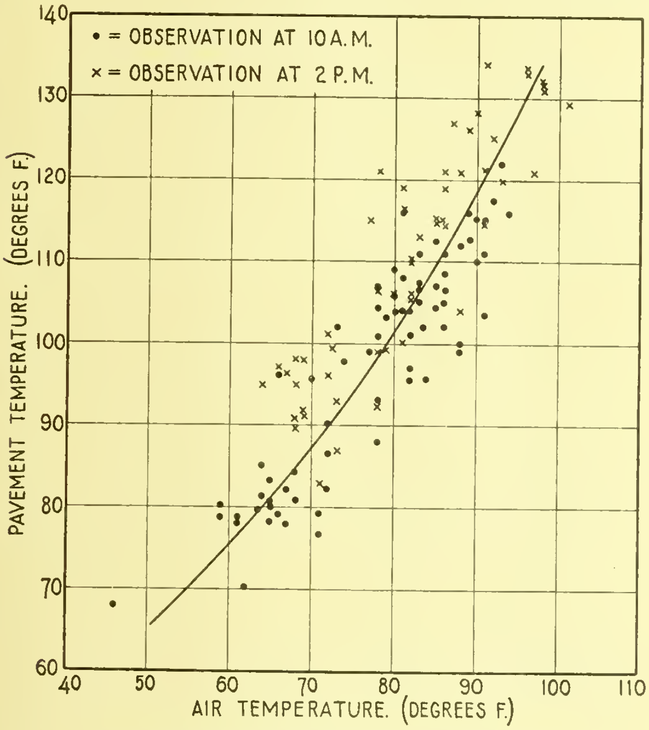


FIG. 3.—Relation between air and sheet asphalt pavement temperature. Observations taken one-half inch below the surface

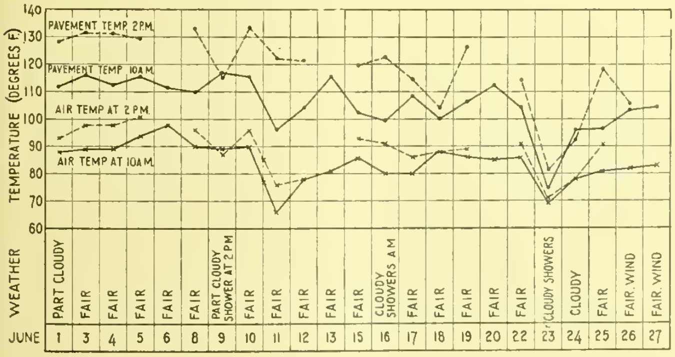


FIG. 4.—Record of corresponding temperatures of air and sheet asphalt pavement one-half inch below the surface

Figure 2 indicates the average movement per 1,000 passages of the truck of all 33 sections of the second series with respect to the average daily air and pavement temperatures for the period or "traffic interval" between which measurements of movement were taken. The movement of each section is determined by averaging the total forward shove of two lines of 25 screws each across the 13-foot roadway. The points plotted on the curve are the averages of the 33 sectional movements. The high degree of plasticity of the pavements during the initial traffic interval ending September 2 may be significant. This period came not only in extremely hot weather, but followed immediately after construction, and it is probable that the apparent instability may be ascribed in considerable degree to the fact that the sections had not attained the ultimate compression to which they were susceptible under traffic. In substantiation of this theory, attention is directed to the fact that virtually equivalent temperatures during the following year resulted in far less displacement. Furthermore, in a considerable number of the mixtures at least 50 per cent of the total movement recorded during the entire test occurred in this short initial period.

By the latter part of the summer of 1925, five sections which carried excessive amounts of bitumen had deformed very badly. In the traveled areas their original internal structure was entirely disrupted and their resistance to displacement at high temperatures was virtually destroyed. The condition of these pavements is reflected in the rather sharp rise in the curve of Figure 2 for the period beginning August 3.

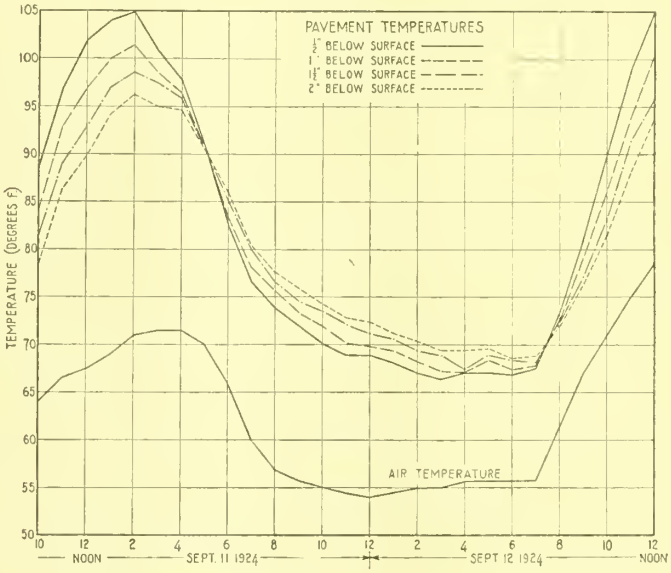


FIG. 5—Hourly record of internal temperature of sheet asphalt pavement

It should be recognized that this chart does not pretend to measure the susceptibility of asphaltic pavements as a class to movement at the indicated temperatures. In these tests many mixtures remained stable at all temperatures whereas others, several of which were obviously of poor design, deformed very badly. Thus the plotted movements are representative only of a group of arbitrarily chosen mixtures.

PAVEMENT TEMPERATURES HIGHER THAN AIR TEMPERATURES

When exposed to direct sunlight the internal temperature of asphaltic pavements is probably always higher than that of the surrounding air. This difference is very likely slight during the winter months, but under summer conditions the bituminous mixture accumulates heat rapidly and its temperature rises faster than that of the air during the period of the day when it is subjected to direct action of the sun's rays. In these tests internal pavement temperatures 25 to 35° F. higher than the air temperatures were of common occurrence. The highest pavement temperature recorded was 140° F. and this was reached only a very few times. Figure 3 shows the relation between air and pavement temperatures, the latter obtained from the thermocouples placed one-half inch beneath the surface. The 10 o'clock and 2 o'clock readings are recorded separately. All plotted points represent data obtained on clear days, although winds of varying intensity and direction often prevailed. It will be noted that, in general, the same air temperature resulted in higher afternoon pavement temperatures indicating that gains by absorption exceeded losses through conduction and radiation.

Rain, clouds, or local shade result in an immediate decrease in the temperature of asphaltic pavements. The effect of the last-named factor was particularly noticeable in several of the mixtures which, during the afternoons of the late summer months, were shaded by a high bank surmounted by trees just west of the test pavement. As the line of shade crossed the location of the embedded thermocouples their temperatures abruptly decreased. In Figure 4 are plotted morning and afternoon observations of air and pavement temperatures, the latter being taken by means of the thermocouple placed one-half inch below the surface of the sheet asphalt mixture. The record extends over a period of approximately four weeks and illustrates characteristic reactions of pavement temperature to changes in climatic conditions.

There appears to be no extreme difference between the temperatures of asphaltic pavements at various depths beneath the surface of the customary 2-inch layer. While the pavement is subjected to direct sunlight the top was found always to be warmest. In these tests, however, differences in temperature at the one-half-inch depth and at the 2-inch depth never exceeded 15° F. On cloudy days the pavement temperatures were virtually uniform throughout, while rain, furnishing a medium for rapid losses of heat from the surface, frequently resulted in slightly higher temperatures at the bottom. This latter condition seems also to prevail at night. Figure 5 shows a series of 26 consecutive hourly temperature measurements taken at four depths in a sheet asphalt pavement.

Of particular interest in this figure are the uniformity of the temperature throughout the pavement at 5 p. m. and 7 a. m., the warmer condition of the protected under layers during the hours of darkness and the quick reversal to normal daytime conditions as direct sunlight once more strikes the pavement the following morning.

In common with all other structures, asphaltic pavements may be so designed that they will prove successful under some conditions but fail under others. Obviously, under no traffic, all 60 of the test sections involved in these experiments would have remained stable regardless of climatic conditions. Test data also show this practically to be the case when traffic was imposed during periods of average air temperature below 70° F. Higher temperatures aided traffic in developing instability in certain mixtures, but with lighter traffic it is certain that several of these would have been classed with those which proved satisfactory. It is clear, therefore, that pavement behavior, frequently explained on the basis of traffic only, should be analyzed also with regard to the prevailing climatic conditions at the time such traffic is imposed.

CONCRETE TESTS TO BE MADE IN NEW JERSEY

The New Jersey State Highway Department, in cooperation with the Bureau of Public Roads, has begun a series of concrete tests for the purpose of studying the relative concrete making properties of crushed stone and gravel used in concrete road construction in that State. The tests are being made in the State Highway Laboratory at Trenton, and involve the fabrication and testing of about 250 concrete beams 8 by 8 by 48 inches in size, as well as a large number of cylinders for compression tests.

The program calls for three series of tests. In the first series the workability of the concrete is to be kept constant, as nearly as possible, by means of the flow test, and the relative yield and strength of the concrete determined for each of several gradations both of crushed stone and gravel, using concrete proportions as given in the current New Jersey Standard Specifications. The object of this series is to determine the relative strength and yield of gravel concrete as compared with crushed stone concrete for several sizes and gradations of coarse aggregate.

In the second series an effort will be made to design concrete of a given strength by means of the water-cement ratio theory, for each type and gradation of coarse aggregate. The procedure to be followed in this series is essentially as follows. To each gradation and type of coarse aggregate, fine aggregate will be added in the following ratios by volume, 33 to 67, 36 to 64 and 40 to 60. To each of the above combinations water and cement in fixed ratio, depending on the strength desired, will be added until the desired workability has been reached. The end point in each case will be determined by means of the flow test, supplemented by the judgment of experienced concrete operators. Concrete specimens will then be made up in the proportions as determined by the trial method referred to, and the comparative strength, which should be constant, the comparative yield, and the comparative absorption will be determined.

In the third series of tests, specimens will be made in which the concrete mixture has been designed in accordance with the fineness modulus theory as given in "The Design and Control of Concrete Mixtures," a publication recently issued by the Portland Cement Association. The results obtained from this series will be used as a check against the results obtained in the second series.

Assuming a constant strength and a constant degree of workability, it is hoped to determine by means of these tests what grading of coarse aggregate and what proportions of fine to coarse will give the greatest yield of concrete for both crushed stone and gravel.

MOTOR VEHICLE REGISTRATIONS, REVENUE, AND GASOLINE TAXES FOR THE YEAR 1925

For 1925 the motor vehicle statistics have been amplified somewhat and are published in two tables; the table on page 50 showing the number and classes of vehicles registered and the table on page 51 showing the corresponding receipts.

REGISTRATION STATISTICS

This table shows by States the motor vehicle registrations, the tax-exempt cars, dealers' licenses issued, and operators' and chauffeurs' permits.

As far as possible all reregistrations are eliminated and to avoid duplication of the recorded cars of other States all nonresident registrations are excluded. The grand total of registered motor cars and trucks has been divided into two classes based on their use, either as passenger-carrying, or commodity-carrying vehicles, the latter including tractors and motor-driven road equipment.

In the first column is the grand total of all registered and taxed motor cars and trucks, which is the total of columns 2 and 3. Column 2 shows the passenger cars, including all privately owned passenger automobiles, taxis and passenger cars for hire, and busses. A few States (noted in column 3) have included busses with motor trucks, but nearly all busses are included in column 2. Column 3 shows the commodity or freight carrying cars, which include all motor trucks, road tractors, and all motorized road vehicles, not primarily used for the transportation of passengers. Tractors used for farm purposes have been excluded although these are registered and pay a fee in a few States. Only 12 States record road tractors separately, amounting in 1925 to 2,749 tractors. The other States include road tractors with trucks.

Considerable interest has been shown in the number of taxis and cars for hire and busses. In the absence of sufficiently complete statistics from registration offices, data have been taken from other sources and shown under the heading "Special list of passenger cars for hire." The list in column 4 taken from Internal Revenue Bureau records shows the number of taxis and passenger cars for hire having seating capacity of two to seven persons for which an occupational Federal tax of $10 was paid by the owners during the fiscal year ending June 30, 1925. A few States supplied this data as of December 31, and where available, it has been used. The list of busses shown in column 5 is taken from the February issue of "Bus Transportation," and is only a partial list. This data has not been used in connection with the official figures summarized in column 2, except in a few States, as noted, which record passenger cars for hire and busses separately.

Registered trailers and motor cycles for which fees are paid are shown. The trailers in column 6 cover semitrailers (two wheels) and trailers (four wheels). In general, trailers are hauled by road tractors and carry commodities or freight. There are some two-wheel trailers used with passenger cars for "camp kits" but these are not segregated in the records. Motor cycles are listed in column 7 and include cycles with or without side cars.

Tax-exempt official cars are shown for all States which record these cars and trucks. The total number of United States cars and trucks given was obtained from the United States Budget Bureau. Under this heading is also shown official motor cycles, which are recorded in less than half the States.

The next to the last column shows the 1924 grand total with 1,696 tractors added to formerly published figures (in States noted) in order to be comparable with the 1925 grand total registered motor cars and trucks.

MOTOR VEHICLE REGISTRATION RECEIPTS AND DISPOSAL OF FUNDS

The first column of this table on page 51 shows the total gross receipts. The next eight columns show various items which make up the gross receipts as segregated by 33 States and the District of Columbia. The States starred in the column of States are the ones which reported the complete details, and these States have been summarized to make a subtotal called "Detailed total." The details of registration receipts correspond with the number of vehicles in the table on page 50 wherever reported. Under "Miscellaneous receipts" are shown dealer's license fees, chauffeurs' and operators' permits and other miscellaneous receipts, the latter covering many items such as reregistration fees, nonresident registration fees, traffic fines (if these are included in the motor vehicle fund) certificates of title, duplicate tags, etc.

The collection and administration expenses of the motor vehicle license offices are generally deducted before final division is made allocating certain shares to the State highways, to county or other local rural roads to pay retirement and interest on State road and bridge bonds, and for other purposes, as noted. Several States as noted pay collection and administration expenses out of State appropriations. In many cases a stated lump sum is authorized for this expense to be deducted from motor vehicle fund, or else a certain percentage of gross receipts is allowed.

GASOLINE TAXES

Data on gasoline tax rates and collections are shown on page 52. This table shows the gross receipts and the net receipts after the deduction of refunds allowed by law. Some States have no provision for refunds and in such cases gasoline used for purposes other than motor vehicles is included. Collection costs are shown where paid out of gasoline tax earnings.

Motor-vehicle registrations for the year 1925

State	Registered motor vehicles, individually and commercially owned			Special list of passenger cars for hire [1]		Other registered vehicles		Tax-exempt official vehicles and motor cycles [2]			Number of licenses, or permits (autos.)			Grand total registered motor cars and trucks, 1924 [3]	Per cent increase 1925 over 1924
	Grand total registered motor cars and trucks, 1925	Passenger automobiles, taxis, and busses	Motor trucks and road tractors	Taxis, etc.	Busses	Trailers	Motorcycles	United States cars	State and local cars	Motorcycles (official)	Dealers [4]	Operators	Chauffeurs		
Alabama	194,580	171,387	23,193	2,710	778	480	524				2,599		2,105	157,262	23.7
Arizona	68,029	59,798	8,231	[5] 568	569		310		759	49	330		339	57,828	17.6
Arkansas	183,589	159,511	24,078	[5] 2,398	378	918	263		572	30				141,983	29.3
California	1,440,541	1,225.796	214,745	5,210	4,017	27,542	11,177		[6] 18,647	583	11,977		106,230	1,319,394	9.2
Colorado	240,097	221,513	18,584	2,416	816	82	1,862				3,206	20,079	7,776	213,247	12.6
Connecticut	250,669	213,486	37,183	[5] 2,301	904	332	3,886		3,139	148	4,386			[7] 217,236	15.4
Delaware	40,140	32,550	7,590	192	138	166	375				763	40,841	3,555	35,136	14.2
Florida	286,388	237,435	49,953	[5] 3,356	1,253	1,062	1,200				2,016		5,656	195,128	46.8
Georgia	248,093	217,578	30,515	1,969	715		994				727		2,921	207,688	19.4
Idaho	81,506	73,896	7,610	558	570	168	518		1,050		324		448	69,227	17.7
Illinois	1,263,177	1,101,943	[8] 161,234	10,374	3,239	3,777	6,603		None.		2,000		100,000	1,119,236	12.8
Indiana	725,410	630,554	94,856	3,648	1,896	5,068	4,525		525		2,242		40,247	651,705	11.3
Iowa	659,202	613,412	45,790	2,284	1,321	125	2,303		2,500	90				616,128	7.0
Kansas	457,033	409,968	[8] 47,065	2,140	696		1,434		2,014					410,891	11.2
Kentucky	261,647	235,020	[8] 26,627	2,981	1,120		703		1,169		1,119		8,867	229,804	13.8
Louisiana	207,000	176,000	31,000	1,477	672		520						10,000	178,000	16.3
Maine	140,499	116,229	24,270	[5] 2,716	376	790	1,293		882	88	1,070	162,435	6,150	[7] 127,598	10.1
Maryland	234,247	222,173	12,074	[5] 3,477	636	586	4,619					109,747	38,185	[7] 198,465	18.0
Massachusetts	646,153	554,813	91,340	6,254	1,857	702	9,401		800	400	2,011	698,378	([9])	570,578	13.2
Michigan	989,010	885,524	[8] 103,486	3,325	2,161	10,592	3,387		3,353		1,958	197,547	75,621	867,545	14.0
Minnesota	569,694	524,879	44,815	1,833	932	1,912	2,923			21	1,943			503,437	13.1
Mississippi	177,262	159,134	18,128	1,555	2,049		100							134,680	33.6
Missouri	604,166	543,426	60,740	4,821	1,407	1,087	1,984		1,317	5				540,500	11.8
Montana	94,656	82,135	12,521	583	323		252		1,000					79,695	18.8
Nebraska	338,719	301,716	[8] 37,003	1,256	323	807	1,207							308,715	9.7
Nevada	21,169	18,069	3,100	137	175	20	120		393	13				18,118	16.8
New Hampshire	81,498	72,472	9,026	1,903	635	497	1,701		300		507	60,772	31,903	[7] 71,149	14.5
New Jersey	580,554	469,156	111,398	5,367	2,401	1,389	7,730		4,469	771				[7] 504,470	15.0
New Mexico	49,111	47,470	1,641	391	308	88	209			3				41,680	17.8
New York	1,625,583	1,346,665	278,918	26,079	3,966	5,051	18,642		10,588	1,192	4,703			1,412,879	15.0
North Carolina [10]	340,287	311,384	28,903	2,102	2,446	500	863		4,110					302,232	12.6
North Dakota	144,972	133,791	[8] 11,181	375	1,078		443							117,346	23.5
Ohio	1,346,400	1,179,400	167,000	5,354	4,103	9,000	12,650		4,200					1,241,600	8.4
Oklahoma	424,345	393,047	31,298	2,212	1,231		817							369,903	14.7
Oregon	216,553	199,517	17,036	732	698	([11])	2,547				598	51,084	15,188	192,615	12.4
Pennsylvania	1,330,433	1,149,074	181,359	6,937	2,615	2,821	15,234		9,750	888	24,105	1,570,219	([9])	[7] 1,228,845	8.3
Rhode Island	101,756	84,337	17,419	1,431	[12] 262	59	1,343		458	64		117,252	([9])	95,482	6.6
South Carolina	168,496	153,343	15,153	1,646	473	824	173		1,261	51				161,753	4.2
South Dakota	168,028	154,141	[8] 13,887	452	205		345		763		959			142,396	18.0
Tennessee	244,626	221,712	22,914	2,301	720		627			50	516			204,680	19.5
Texas	975,083	886,362	88,721	6,454	1,260	4,600	2,228		1,302	551				[7] 801,833	21.6
Utah	90,500	79,170	11,330	312	460	200	719		1,015					68,316	32.5
Vermont	69,576	64,566	5,010	1,134	153		718							61,179	13.7
Virginia	282,650	216,950	35,700	2,543	1,373	440	1,590		2,435	125				261,945	7.9
Washington	328,442	281,452	46,990	[5] 1,633	1,574	1,595	2,879			141				295.443	11.1
West Virginia	217,589	190,257	27,332	1,569	856	345	1,432				7,700	64,702	26,648	[7] 191,085	13.9
Wisconsin	594,386	528,090	[8] 66,296	2,535	820	([11])	3,443		555	80	2,700			525,221	13.2
Wyoming	47,711	42,547	5,164	347	553		220		203					43,639	9.3
District of Columbia	103,092	89,790	13,302	1,182	265	No fee.	1,312					17,503	3,126	88,762	16.1
Totals	19,954,347	17,512,638	2,441,709	145,530	[1] 57,826	83,625	140,348	[13] 17,400	79,529	5,343				[7] 17,593,677	13.4

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[1] Special list from sources other than registration offices. Taxis and cars for hire taken largely from Internal Revenue Bureau report. List of busses taken largely from "Bus Transportation." These vehicles are included in the grand total given in the first column.

[2] Not included in the grand total given in the first column.

[3] Total formerly published has been revised to include 1696 road tractors to give a figure comparable to the 1925 total which for the first time includes road tractors.

[4] As reported by States and is not complete.

[5] As reported by motor vehicle bureau.

[6] Includes 7,728 Public Service Corporation vehicles which are tax exempt.

[7] Includes road tractors formerly not included.

[8] Includes busses as reported by State.

[9] Included with operators licenses.

[10] Only data from July 1 to Dec. 31 reported.

[11] Included with motor trucks and tractors.

[12] Total reported by State which is larger than that shown in "Bus Transportation."

[13] From records of the Bureau of the Budget. War Department vehicles not included.

Receipts from motor vehicle registration fees, etc., for the year 1925

States	Total gross receipts[1]	Subdivision of registration receipts[2]					Miscellaneous receipts			Disposition of gross receipts				
		Motor cars			Other vehicles						For highway purposes			
		Total motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor cycles	Dealers' licenses	Chauffeur and operator permits	Miscellaneous	Collection and administration	State highways	Local roads	State road bonds	For other purposes
Alabama	$2,511,129	[3] $2,494,820					$2,599	$10,410	$3,299	[4] $105,527	$769,874	$486,490	$1,138,828	[5] $10,410
Arizona	405,592	385,032				$644	3,649	1,695	14,573	18,000	[6] 387,592			
Arkansas	3,150,000	(7)								12,000	1,731,000	583,000	824,000	
California*	7,816,298	6,754,002	$4,081,130	$2,672,872	$209,185	39,956	42,251	258,684	512,220	951,076	3,432,611	3,073,607		[8] 359,004
Colorado*	1,430,299	1,336,392	1,127,149	209,243	1,140	3,724			89,043	71,515	679,392	679,392		
Connecticut*	5,644,247	4,303,483	3,178,878	1,124,605	7,853	15,376			1,317,535		5,644,247			
Delaware*	680,700	517,004	378,265	138,739	2,269	1,485	7,990	133,720	18,232		680,700			
Florida*	3,645,628	3,449,052	2,536,383	912,669	18,927	4,803	24,435	9,019	139,392	261,220	2,538,306	846,102		
Georgia*	3,010,415	2,952,609	2,473,485	479,124		4,081	42,700	5,594	5,431	98,297	2,912,118			
Idaho*	1,192,587	1,155,174	967,860	187,314	3,711	2,450	19,515	896	10,841	(9)	140,444	1,037,226	14,917	
Illinois*	12,969,754	12,111,679	9,259,929	2,851,750	46,004	23,963	88,050	355,519	344,539	(10)	9,982,450		2,987,304	
Indiana*	4,649,663	4,318,734	3,300,396	1,018,338	17,352	8,336	53,950	74,567	176,724	205,681	4,443,982			
Iowa	9,741,103	(7)								713,036	5,758,141		3,030,325	[11] 239,601
Kansas	4,610,090	(7)								230,505	3,284,689	1,094,896		
Kentucky*	3,780,062	3,664,979	2,864,448	800,531		5,581	31,012	16,819	61,671	132,105	3,247,733	400,224		
Louisiana	3,400,045	3,343,049				2,600		54,396		40,000	3,360,045			
Maine*	2,182,135	1,671,096	1,330,814	340,282	2,615	7,936	324,870	32,952	142,666	[12] 254,526	1,302,196		552,647	[13] 72,766
Maryland*	2,576,301	2,006,322	1,744,423	261,899	11,978	15,682		262,595	279,724	[14] 250,000	2,326,301			
Massachusetts*	9,843,901	7,346,952	5,794,224	1,552,728	14,795	47,069	59,700	1,396,756	978,629	921,514	8,922,387			
Michigan*	14,526,002	13,107,863	10,160,579	2,947,284	121,435	13,234	86,563	241,782	955,125	300,000	7,356,467	6,000,000		[15] 869,535
Minnesota*	9,744,834	9,651,795	8,654,290	997,505	6,847	11,743	34,092		40,357	(16)	6,294,834		3,450,000	
Mississippi*	1,530,000	1,529,150	1,377,000	152,150		850				45,900		1,484,100		
Missouri	7,267,098	(7)								432,023	6,835,075			
Montana*	915,253	914,878	788,125	126,753		375				32,000		883,253		
Nebraska*	3,936,458	3,791,628	3,141,477	650,151	3,456	4,902			136,472	98,411	1,151,414	2,686,633		
Nevada	209,197	208,401				600			196	10,584	114,225	3,138	81,250	
New Hampshire	1,736,094	1,383,969			(17)	9,556	28,401	229,535	84,633	114,610	1,613,804			[18] 7,680
New Jersey*	10,515,323	7,582,255	4,527,893	3,054,362	45,895	15,460	63,661	1,983,948	824,104	1,177,057	5,552,266	3,725,000		[19] 61,000
New Mexico*	457,874	447,001	403,344	43,657	570	728			9,575	31,991	283,922	141,961		
New York*	25,506,245	22,502,688	15,675,072	6,827,616	36,168	85,186	153,745		2,728,458	[20] 372,848	18,876,461	6,241,060		[18] 15,876
North Carolina	[21] 8,359,844	(7)								149,761	8,210,083		(22)	
North Dakota*	1,083,573	1,049,324	935,031	114,293		1,397			32,852	150,000	401,787	401,786	130,000	
Ohio	13,147,231	(7)								(23)	6,573,616	6,573,615		
Oklahoma	4,576,572	(7)								(24)	187,858	3,978,022		[25] 410,692
Oregon*	5,370,202	5,207,691	4,440,577	767,114	(17)	14,629	17,570	77,107	53,205	200,000		1,292,551	3,877,651	
Pennsylvania*	21,926,972	16,934,504	11,568,692	5,365,812	29,277	41,932	296,887	1,721,187	2,903,185	[26] 2,563,137	18,952,448			[27] 411,387
Rhode Island*	1,863,955	1,432,561	1,059,054	373,507	1,003	5,009	13,340	234,504	177,538	306,492	1,557,463			
South Carolina*	2,366,076	2,106,271	1,784,735	321,536	13,710	1,567	25,670	1,141	217,717	187,729	1,736,716			[28] 441,631
South Dakota*	2,445,112	2,403,501	2,143,944	259,557		1,630	23,975		16,006	21,511	1,222,556	1,201,045		
Tennessee	3,060,948	(7)								54,243	3,006,705			
Texas	13,477,931	8,976,151				11,140			4,490,640	476,146	9,368,187	3,633,598		
Utah	554,235	(7)											[29] 554,235	
Vermont*	1,497,146	1,265,611	1,145,126	120,485		5,000			226,535	82,037	1,415,109			
Virginia*	4,300,950	3,947,402	3,414,997	532,405	4,594	7,576			341,378	(30)	4,122,018			[31] 178,932
Washington*	4,980,026	4,848,572	3,774,828	1,073,744	32,715	15,414			83,325	240,059	4,665,195	74,772		
West Virginia*	3,354,247	3,022,617	2,470,524	552,093	2,577	5,902	40,910	137,620	144,621	264,386	783,573		2,000,000	[32] 306,288
Wisconsin*	7,896,210	7,659,722	6,309,848	1,349,874	(17)	21,140	86,775		128,573	380,000	5,626,210	1,875,000	(33)	[19] 15,000
Wyoming*	482,857	470,459	378,169	92,290		1,054			11,344	(10)			482,857	
District of Columbia*	291,207	111,758	98,456	13,302		1,312		49,809	128,328	[34] 36,820	[35] 254,387			
Detailed total[2]	184,412,512	161,574,729	123,289,145	38,285,584	634,076	436,482	1,537,661	6,994,219	13,235,345					
Grand total	260,619,621									11,992,747	177,706,587	48,396,471	19,124,014	3,399,802

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[1] Total funds derived from operation of motor-vehicle laws, including registration fees, licenses, permits, fines, etc.

[2] Only the 33 States and the District of Columbia reported data in full detail, 6 gave partial details and 9 gave no details, therefore the detailed total is less than that of the first column.

[3] Includes all registered vehicles.

[4] Includes $62,370 for probate judges.

[5] Amount from licenses of taxi chauffeurs allotted to State general fund.

[6] For maintenance work.

[7] No details given.

[8] Traffic officers' expenses, deducted from county's share of net receipts.

[9] Special State appropriation through State highway fund.

[10] Special State appropriation.

[11] For State highway commission maintenance.

[12] Includes $153,531 for motor-vehicle law enforcement.

[13] Expenses of State highway commission.

[14] Estimated.

[15] Expenses of motor-vehicle theft department.

[16] Estimated at $302,600 paid from State appropriation

[17] Included under motor cars.

[18] Refunds.

[19] Toll bridge commission.

[20] Collection fees of county clerks in addition to the expenses of 7 city offices, $1,857,900, taken from general State fund.

[21] For period of 6 months, July 1 to December 31, as registration year begins July 1..

[22] Interest and sinking fund requirements included in State highway amount.

[23] Special legislative appropriation of $363,659.

[24] Expenses from State highway department fund.

[25] State general fund to July 1, 1925; not to receive any share after this date.

[26] $1,420,048 expended for administration and balance for administration of road work by State highway department.

[27] For State highway patrol.

[28] Includes $374,140 refund by amendment to law and $67,491 to State genera fund.

[29] Includes amount spent on collection and administration.

[30] State appropriation of $296,969.05.

[31] Operation of auto theft law.

[32] State road commission expenses.

[33] Bond payments included with other items.

[34] All money collected deposited in U. S. Treasury. This amount is the appropriation for expenses of administration.

[35] Amount to balance with gross receipts. The United States appropriations for streets is much higher.

Gasoline taxes for the year 1925

States	Gross tax assessed, prior to deduction of refunds	Exemption refunds: (Deduct from gross tax)	Total tax earnings on fuel for motor vehicles	Disposition of total tax earnings				Tax rates, 1925			Net gallons of gasoline taxed and used by motor vehicles	Estimated additional gallons (not taxed) used by motor vehicles
				Collection costs	Construction and maintenance of rural roads		For other purposes	Cents per gallon		Date of rate change		
					State highways	Local roads		Jan. 1	Dec. 31			
Alabama	$2,140,802		$2,140,802	$9,461		[1] $2,131,341		2	2		107,040,092	
Arizona	1,035,551	$179,600	855,951		$427,976	427,975		3	3		28,531,686	
Arkansas [2]	3,230,559	280,199	2,950,360		1,357,360	[3] 1,593,000		4	4		73,759,002	
California	16,150,387	1,193,598	14,956,789	7,393	7,229,248	7,229,248	[4] $490,900	2	2		747,839,462	
Colorado	1,991,531	30,585	1,960,946		980,473	980,473		2	2		97,377,858	
Connecticut	1,908,809		1,908,809		1,908,809			1	2	July 1	122,230,292	
Delaware	350,580	8,499	342,081		342,081			2	2		17,104,050	
Florida	7,657,507		7,657,507	6,000	5,549,978	2,101,529		3	4	June 6	210,323,517	
Georgia	4,418,824		4,418,824	4,200	1,641,248	1,386,688	[5] 1,386,688	3	3½	Aug. 26	138,802,152	
Idaho	932,064	36,621	895,443	9,466	885,977			2	3	Mar. 1	30,809,320	
Illinois	None.							0	0	No tax.		530,534,340
Indiana	7,832,462	179,413	7,653,049	12,436	5,200,637	2,439,976		2	3	Apr. 1	272,980,870	
Iowa	3,568,184	63,069	3,505,115	5,520	1,151,144	2,302,289	[6] 46,162	0	2	Apr. 16	175,255,740	51,796,350
Kansas	3,000,253	95,059	2,905,194		2,905,194			0	2	May 1	145,259,690	52,911,710
Kentucky	3,041,560		3,041,560		3,041,560			3	[7] 3		101,385,318	
Louisiana	2,339,543		2,339,543		[8] 2,339,543			2	2		116,939,139	
Maine	1,283,874	15,526	1,268,348	5,596	[9] 1,262,752			1	3	July 11	56,513,741	
Maryland	2,022,986	45,950	1,977,036	2,500	[10] 1,579,629		[11] 394,907	2	2		98,851,813	
Massachusetts	None.							0	0	No tax.		274,615,025
Michigan	8,742,392	506,314	8,236,078	41,358	[12] 6,694,720	[13] 1,500,000		0	2	Feb. 1	411,803,894	19,856,370
Minnesota	3,989,282	125,342	3,863,940		3,863,940			0	2	May 1	199,464,097	57,908,930
Mississippi	2,494,274		2,494,274	1,800	[8] 1,224,976	[8] 1,203,715	[14] 63,783	3	3		83,142,469	
Missouri	4,234,070	74,955	4,159,115	23,429	4,135,686			2	2	Jan. 1	207,955,474	
Montana	674,710		674,710		101,207	371,090	[15] 202,413	2	2		33,735,497	
Nebraska	2,202,236	8,434	2,193,802	4,963	2,188,839			0	2	Apr. 1	109,690,122	26,570,900
Nevada	335,446	16,741	318,705		159,353	159,352		2	4	do	8,850,407	
New Hampshire	716,140	9,068	707,072		707,072			2	2		35,353,585	
New Jersey	None.							0	0	No tax.		249,638,220
New Mexico	537,356		537,356	26,868	[8] 510,488			1	3	Mar. 17	20,490,892	
New York	None.							0	0	No tax.		715,256,520
North Carolina	6,238,508	156,130	6,082,378		6,082,378			3	4	Feb. 21	161,371,522	
North Dakota	649,416	15,000	634,416		224,095		[15] 410,321	1	1		64,941,557	
Ohio	9,133,785	123,835	9,009,950		[8] 4,054,478	[8] 2,252,487	[16] 2,702,985	0	2	Apr. 18	450,497,522	99,560,500
Oklahoma	5,143,517		5,143,517		3,351,898	1,791,619		2½	3	Mar. 23	176,753,177	
Oregon	3,065,151	156,056	2,909,095	6,553	2,902,542			3	3		96,969,835	
Pennsylvania	8,352,798		[17] 8,352,798		3,136,819	2,105,917	[15] 3,110,062	2	2		414,096,490	
Rhode Island	318,357		318,357		318,357			0	1	Apr. 29	31,835,668	3,576,640
South Carolina	3,870,588	5,185	3,865,403		2,186,152	1,512,889	[18] 166,362	3	5	Mar. 23	83,962,562	
South Dakota	2,122,406	274,808	1,847,598		1,847,598			2	3	Mar. 10	64,024,928	
Tennessee	3,407,886		3,407,886	22,768	3,385,118			2	3	Feb. 9	122,000,680	
Texas	4,641,784		4,641,784		3,481,338		[19] 1,160,446	1	1		464,178,427	
Utah	1,064,004		1,064,004	3,750	[20] 1,060,254			2½	3½	Apr. 1	32,217,216	
Vermont	502,272		502,272		[8] 502,272			1	2	Feb. 26	25,863,167	
Virginia	3,863,117	161,166	3,701,951	5,604	2,464,231	1,232,116		3	[21] 3		123,398,365	
Washington	3,205,114	184,302	3,020,812		3,020,812			2	2		151,040,586	
West Virginia	2,222,329	35,590	2,186,739	7,500	[22] 2,179,239			2	3½	July 1	76,331,660	
Wisconsin	4,155,469	123,793	4,031,676	10,000	4,021,676			0	2	Apr. 1	201,583,789	48,830,860
Wyoming	460,972	4,675	456,297	228	456,069			1	2½	do	20,746,056	
District of Columbia	896,568	6,970	889,598				[23] 889,598	2	2		44,479,898	
Total			146,028,940	217,393	102,065,216	32,721,704	11,024,627	Av. 2.26			6,457,783,284	2,131,056,365

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[1] For maintenance only.
[2] In addition $438,436 collected as motor oil tax at a rate of 10 cents per gallon.
[3] Includes $873,240 payments on county road and bridge bonds.
[4] Delinquent taxes uncollected not disposable in 1925.
[5] To State treasury.
[6] Unaccounted for; probably delinquent taxes.
[7] Tax increased to 5 cents effective February 21, 1926.
[8] For maintenance only.
[9] Includes $282,913 for maintenance.
[10] For maintenance and reconstruction.
[11] For maintenance of Baltimore streets.
[12] Includes $3,000,000 for interest and retirement payments on State road bonds.
[13] Payments to counties on State award highways.
[14] For sea-wall in Harrison County.
[15] For State general fund.
[16] Maintenance of municipal streets.
[17] Includes $70,868 paid-in delinquent taxes of former years.
[18] Covers part of first four months of year only, as new law excludes State general fund from share in gasoline tax fund.
[19] For free school fund.
[20] Includes $460,000 payment of interest and to sinking fund on State road bonds.
[21] Tax increased to 4½ cents effective Mar. 11, 1926.
[22] Includes $1,520,463 payment of interest on State road bonds.
[23] For improvement and repair of Washington streets.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*724. Drainage Methods and Foundations for County Roads. 20c
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94. TNT as a Blasting Explosive.
331. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60. Federal Legislation Providing for Federal Aid in Highway Construction.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.
914. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MARCH 31, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 6,970,097.71	$ 2,863,197.86	611.8	$ 10,736,200.15	$ 6,141,722.18	588.9	$ 6,008,211.96	$ 2,854,687.23	287.8	$ 133,112.16	$ 66,656.08		$ 3,423,391.66	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	1,139,122.72	706,313.84	98.4	1,151,770.40	757,869.71	82.8	176,977.10	108,150.70	26.7	3,028,794.81	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	4,278,673.15	1,904,193.17	222.9	5,310,982.30	2,555,646.63	345.6	536,670.32	236,160.00	37.4	1,629,622.67	Arkansas
California	22,346,175.99	10,719,249.61	894.8	3,136,696.93	1,471,494.66	112.7	12,290,177.69	6,035,504.13	361.3				3,846,566.71	California
Colorado	11,876,703.94	6,067,814.34	651.2	1,151,021.44	598,197.01	54.6	4,517,663.22	2,278,314.49	234.7	225,774.02	126,626.34	12.6	3,254,860.82	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	750,965.91	250,212.14	13.4	1,901,818.16	690,259.68	29.1	582,417.39	144,362.95	9.1	1,529,477.57	Connecticut
Delaware	4,281,559.81	1,496,190.66	107.1	456,744.13	212,544.95	12.3	980,042.66	388,784.75	22.7	671,272.05	59,220.00	13.8	317,317.65	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				10,346,486.92	4,918,554.60	296.4	130,171.16	65,085.58	13.7	1,695,826.85	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	3,584,313.71	1,743,792.54	232.3	11,360,710.24	5,599,100.33	689.2	1,210,588.87	600,876.20	56.6	1,081,817.47	Georgia
Idaho	9,394,676.80	4,816,332.26	600.1	1,463,421.38	912,273.25	103.5	2,018,134.62	1,233,556.87	127.2	644,306.66	357,911.76	16.8	1,240,563.86	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	3,012,732.19	1,447,994.11	99.6	6,344,557.34	3,068,862.83	218.0				6,676,264.78	Illinois
Indiana	13,639,172.66	6,562,455.68	422.1	2,144,221.16	1,035,593.69	73.2	17,087,806.75	8,003,335.26	464.7	421,901.51	208,509.47	12.8	2,394,460.90	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	722,462.52	320,365.05	36.5	8,105,894.60	3,660,339.59	498.3	3,541,207.32	1,683,812.99	244.2	2,713,652.37	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	5,319,314.12	2,349,067.90	286.1	10,642,830.38	4,128,274.72	539.9	1,991,243.72	625,729.10	127.9	2,606,065.96	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	3,521,723.17	1,381,242.75	96.4	7,187,922.44	3,276,846.30	308.8	257,788.45	111,997.22	10.2	2,236,728.14	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,622,969.92	736,731.50	103.4	3,420,155.43	1,659,669.90	160.0				1,597,135.74	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	673,271.45	284,537.06	22.2	1,809,495.54	706,814.24	57.6	92,205.65	46,102.82	4.7	1,619,403.66	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,711,853.25	1,234,208.07	126.9	249,190.03	114,397.50	12.7				727,068.28	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	2,198,203.20	591,531.01	40.1	4,484,811.18	1,267,266.18	60.7	1,161,802.28	285,284.20	16.9	2,496,983.33	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,286,862.11	4,283,204.37	335.7	9,147,721.14	4,128,580.06	218.0	382,294.00	178,607.13	17.2	4,423,656.53	Michigan
Minnesota	30,415,686.89	12,738,642.04	2,721.2	5,271,152.99	2,284,974.52	397.1	7,803,579.46	2,495,300.00	522.2	1,288,037.91	314,000.00	71.7	1,758,863.44	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	3,000,939.78	1,499,508.16	221.4	8,026,600.06	4,007,044.36	419.8	685,436.85	283,918.53	36.0	1,348,844.22	Mississippi
Missouri	17,368,156.57	8,219,411.43	1,118.9	8,614,993.14	4,162,082.75	311.6	20,833,047.86	8,338,598.72	586.1	1,776,184.96	788,225.64	68.0	1,278,117.46	Missouri
Montana	10,156,600.41	5,317,623.15	921.6	1,244,383.40	1,015,942.74	133.3	1,380,058.51	930,089.13	107.3	945,723.12	513,821.09	110.2	5,647,508.89	Montana
Nebraska	9,306,374.36	4,389,623.50	1,570.6	1,521,442.87	736,793.25	120.3	11,107,158.19	5,502,447.60	1,081.1	2,065,372.88	1,030,828.86	206.2	2,976,641.79	Nebraska
Nevada	4,917,466.69	3,088,299.78	357.3	2,029,124.24	1,587,127.57	160.8	3,678,708.88	3,097,062.10	335.3				1,022,726.55	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	781,097.31	369,244.23	27.9	632,628.69	296,233.16	19.2	45,925.79	22,962.89	1.4	494,824.85	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	2,683,712.79	726,976.44	47.4	7,872,904.26	2,948,900.47	44.1	420,870.40	66,450.00	4.4	904,413.10	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	3,545,064.81	2,313,941.49	329.6	1,483,968.37	978,752.14	116.4	74,555.37	46,757.90	5.4	2,718,863.86	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	9,095,913.25	3,668,925.81	230.8	31,442,495.58	9,257,492.24	586.5	8,313,370.00	2,009,390.00	134.4	6,880,310.42	New York
North Carolina	21,014,450.41	8,746,464.59	1,119.8	3,923,647.63	1,599,863.21	94.0	8,456,432.34	3,619,780.38	204.2	870,202.76	435,101.38	14.9	1,316,006.44	North Carolina
North Dakota	10,829,263.82	6,268,930.47	1,917.6	1,420,744.22	731,277.64	275.6	3,482,106.39	1,723,110.61	426.5	1,393,357.25	723,263.22	162.7	2,302,077.16	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	6,059,391.44	2,107,214.63	171.8	8,342,639.32	3,301,752.39	264.2	2,788,407.93	906,308.43	70.3	4,171,526.62	Ohio
Oklahoma	20,787,024.94	9,672,890.34	862.2	4,319,668.63	2,006,425.89	174.3	5,371,626.48	2,569,148.85	242.2	234,268.05	110,765.51	7.3	1,710,556.41	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,325,783.62	1,256,645.21	111.3	2,290,319.28	1,378,618.00	119.0	1,434,441.46	659,928.75	46.7	441,790.41	Oregon
Pennsylvania	43,054,835.18	16,222,023.97	850.3	10,228,991.33	3,033,594.88	187.0	29,751,774.10	8,026,969.22	557.8	5,206,220.69	1,668,106.77	124.6	2,388,086.16	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,307,118.37	412,640.21	21.9	1,586,883.95	454,596.67	28.5	46,705.65	13,050.00	0.9	667,495.13	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.9	1,345,670.54	504,380.22	71.3	7,472,425.62	3,330,372.90	377.4	435,848.65	177,606.98	33.2	667,895.76	South Carolina
South Dakota	12,091,434.57	5,989,879.00	1,447.9	4,468,181.03	2,162,037.09	596.0	3,521,652.42	1,680,790.49	554.8	89,856.30	44,928.15	18.0	1,289,155.27	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	6,210,462.31	2,761,448.99	211.9	7,868,816.05	3,661,034.16	280.4	1,246,282.97	594,167.54	38.0	1,531,860.55	Tennessee
Texas	54,120,970.83	21,057,940.12	3,907.1	12,224,766.29	5,120,935.54	736.6	19,738,316.97	8,826,167.77	1,114.8	3,067,800.41	1,379,032.93	121.6	4,223,364.64	Texas
Utah	6,259,159.41	3,818,836.91	423.1	1,034,736.98	536,446.44	52.1	2,630,523.20	1,896,195.08	226.2	32,351.34	23,939.99	0.1	1,443,360.58	Utah
Vermont	3,015,174.51	1,452,894.45	107.8	1,186,757.35	544,749.67	26.4	1,066,065.68	444,520.18	23.5	148,135.36	74,067.67	0.3	752,275.03	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,148,301.82	3,764,679.49	297.5	6,035,474.99	2,596,064.62	187.1	1,473,332.50	704,234.98	39.5	164,536.71	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,591,201.52	1,607,897.59	137.2	2,525,208.65	1,232,400.00	27.6				1,188,266.54	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	1,553,622.90	632,072.94	54.5	5,669,701.47	2,261,429.37	143.9	370,896.18	142,956.79	14.0	1,085,758.57	West Virginia
Wisconsin	21,807,140.91	8,919,640.52	1,451.7	2,044,789.76	1,013,336.87	96.2	4,878,226.54	2,393,891.23	234.0	643,196.13	321,596.00	33.2	4,790,350.28	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	1,845,944.24	1,126,568.38	145.7	2,632,505.19	1,658,909.53	166.0	91,395.64	58,675.00	7.3	983,024.42	Wyoming
Hawaii							1,050,897.93	312,535.18	15.9				787,617.82	Hawaii
TOTALS	740,140,790.22	325,654,346.00	41,898.3	168,826,367.15	75,992,051.60	8,099.5	342,999,099.22	146,435,959.24	14,015.6	47,146,908.88	18,019,076.54	1,990.9	105,273,566.62	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 86,434,928.71 Federal aid $ 36,787,228.02 Miles 3,326.3

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 3 ▽ MAY, 1926

CONTROL OF THE MISSOURI RIVER AT HIGHWAY CROSSINGS IS AN IMPORTANT PROBLEM

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1926

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
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BUREAU OF PUBLIC ROADS

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VOL. 7, NO. 3 MAY, 1926 H. S. FAIRBANK, Editor

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RETARDS IN STREAM CONTROL

Reported by **JOHN R. CHAMBERLAIN**, Highway Bridge Engineer, United States Bureau of Public Roads[1]

IN CONNECTION with the construction of Federal-aid highway bridges across the Missouri River and other streams the Bureau of Public Roads has had occasion to study the problem of stream control. Where a large and relatively permanent bridge is to be placed across a stream, which if uncontrolled is constantly shifting its channel, the problem is one of major importance. A study of the erosion and silting action of such streams must be made in order that bridge sites requiring the least protection may be selected and that adequate protection may be included as a part of the construction plan. There are cases where the cost of holding a stream to the existing channel under a bridge for a period of years has greatly exceeded the cost of the bridge.

CHARACTERISTICS OF THE MISSOURI RIVER

From Sioux City in northwestern Iowa to the mouth of the river at St. Louis the Missouri has an average fall of 0.86 foot per mile. The valley is flat and lies between bluffs 3 to 10 miles apart, with an average width of about 5 miles.

The land in the valley is mostly of a light soil, easily eroded, and in no place does it lie much above flood stage elevation. Bed rock is mostly from 50 to 100 feet below low water and there is little variation in the character of soil to this depth.

The stream carries great quantities of silt in suspension. In August 1923, when unfiltered water from the river was pumped into the Omaha distributing system it was said to contain as high as 43 per cent of silt. This was, of course, considerably above the average.

The stages of the stream are usually referred to as standard high and standard low elevations. These stages are the average high and low for the period under observation prior to 1888. The difference between standard high and standard low at Sioux City is 10.42 feet; at Kansas City, 14.52 feet; and at St. Charles, 16.14 feet. The flood discharge at Sioux City at standard high stage is about 200,000 cubic feet per second and at St. Charles, 300,000 cubic feet per second. The flood of 1892 discharged 650,000 cubic feet per second and that of 1903, 750,000 cubic feet per second at St. Charles.

The width of the stream between banks or standard high water contour varies from about 1,000 feet to more than a mile in places. Its depth at low water is insufficient to float a boat of 4 feet draught though as great a depth as 65 feet has been observed at flood stage in certain places.

The stream, in general, meanders back and forth in the valley from bluff to bluff and, by reason of the rather extreme fall and the instability of the soil it effects, when not controlled, rather pronounced changes in location by continuous erosion of its banks. Maps of this stream made to-day superimposed on maps made prior to 1890 show in some places, particularly in the Dakotas, such erratic changes that no relation can be seen between the location of channel now and then. Figure 1 shows a section of the river below Omaha, Nebr., as it existed prior to 1890 and in 1898. Parts of the river as it is to-day are also shown. It is not unusual for erosion to change the lines of the stream as much as a half mile in a single year.

PROCESSES OF EROSION

Changes in channel occur by overflowing through a swale and thus developing a secondary into a principal channel, and by lateral erosion. The former is the most spectacular but the latter is the cause of the greatest concern. Sometimes at flood stage the overflow will cut across a large horseshoe bend and will erode a channel of river proportions in a very short time and by such action shorten its length many miles. Such a major shortening of the stream gives rise to far-reaching effects. By increasing the slope it produces higher velocity in the stream both up and down for many miles. There also follows a change in the oscillation of the current between banks. The result is excessive lateral erosion tending to cut more and deeper bends. Many such major cut-offs have taken place in

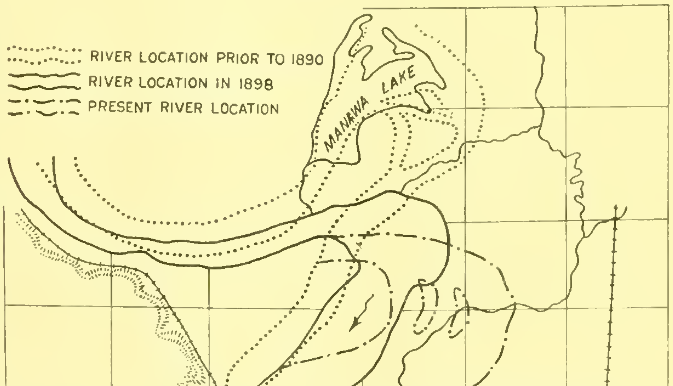

FIG. 1.—Section of the Missouri River below Omaha, Nebr., showing pronounced changes in channel from about 1890 to the present time

recent years and the stream has corrected itself by lateral erosion so that its total length between distant points has remained practically unchanged.

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[1] This report was prepared by Mr. Chamberlain a few months prior to his untimely death on Dec. 15, 1925.

In general the river is a series of bends first to the right and then to the left. In these bends the water is deep along the concave bank and unless the concave bank coincides with the rock bluff at the edge of the valley or is protected by artificial means the shore line yields to erosion.

Even in places where the general direction of the river is straight for a considerable distance, lateral erosion may develop if conditions have been such as to form a bar in midstream, a condition which will normally occur where the river is wide or just below a reach where velocity is great. The bar in the path of the current has the effect of crowding the stream to the sides where it attacks the banks. This was the condition just above Glasgow, Mo., when extreme erosion made expensive protection necessary to save the Chicago & Alton railroad in 1924. It was also probably the principal cause of rapid cutting of the north bank at Waverly in 1923. Instead of the bar itself yielding to the attack of the current, presumably because it presents a taper or wedge to the current, it deflects the stream without itself suffering dislodgment of material. Examples of this condition may be noted in Figure 2 where protection work is shown along the convex shore at East Omaha.

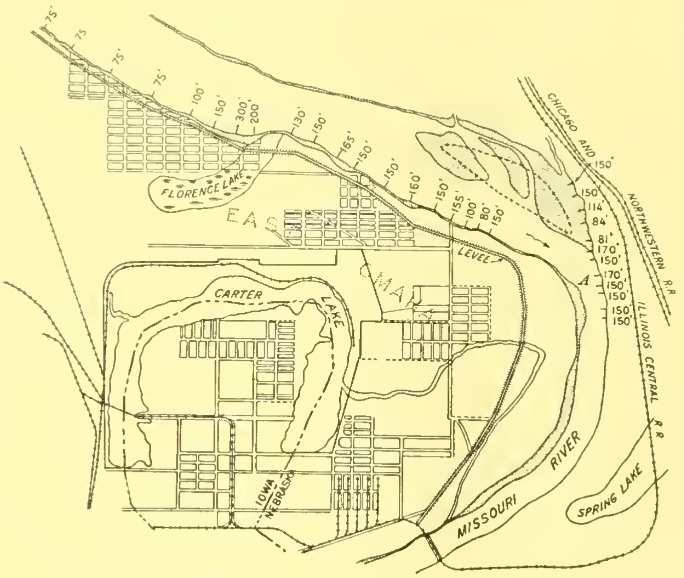

FIG. 2.—Map showing location of retards at East Omaha, Nebr.

At points where bank erosion is severe, excessive depths are usually found and the erosion is most active during a receding stage, probably because the soil is then soaked and the ground water presents a reverse head tending to dislodge the particles of soil in the bank. As great a depth as 65 feet has been observed along such eroding banks at a point about 100 feet off shore. With this in mind the difficulties of placing any kind of construction to stop erosion can be appreciated.

As a bend develops and embodies a change of direction or central angle of near 180° there comes a time when the fall round the convex shore line is sufficiently greater than around the outside or concave shore line to cause the major part of the flow to follow the shorter passage and thus lessen the current near the concave bank. This is referred to as chord action, as the current flows in the direction of a chord and, continuing in a straight line across its former channel, impinges directly against the outside bank. When this occurs it presents a difficult problem, in bank protection. Such a condition exists near East Omaha the current impinging against the bank at the point marked *A* in Figure 2.

METHODS EMPLOYED IN STREAM CONTROL

There are two general methods used in the control of streams. The first is to change the flow by directing the current away from the eroding bank in a desired direction. The other is to accept the current as found and make the bank safe against erosion. The two methods are sometimes combined by retarding the current along the eroding bank and at the same time partially deflecting it away. As an incidental result a deposit of silt is formed on the downstream side of the obstruction used for retarding and this in turn results in building out the bank to some new shore line.

Where the river lies close to a rock bluff or edge of valley or where it can be made to do so the scheme of so deflecting the current that it will maintain such a position is often a desirable undertaking and if the alignment of the rock bluff is straight or slightly concave, it becomes a relatively easy task to train the stream by deflecting dikes. This practice is particularly recommended for bridge crossings of streams that must be kept open to navigation.

Where the stream forms a bend in mid valley and the bend is relatively smooth and on a flat curve that gives promise of future stability if maintained the second method is ordinarily best adapted.

Retarding and partially deflecting the current appears to be the method most commonly used by land owners presumably because it is the only scheme that lends itself to minor operations. If permeable dikes are introduced at intervals along the shore line, the water is retarded and if the velocity is slower after having passed the obstruction, it follows that the excess must be deflected out around the end and this deflection of current tends to change the direction of the stream at that point. Thus if a prism of water approaches the obstacle with a velocity of 4 feet per second and that portion which passes is reduced to 2 feet per second one half of the flow will of necessity be deflected. It also follows that if the space behind the obstruction is eventually filled with silt, the only remaining function of the obstruction is to deflect.

Making the bank proof against erosion has the advantage over the deflection of current in that it precludes the possibility of injuring other property owners along the stream.

If work is installed on a piecemeal basis or in isolated projects instead of over long reaches, its success is threatened by changes that may occur up stream no matter what methods are used. If the work has been placed to meet conditions as they exist, such as the protection of a bend, and if the point of attack of the stream changes so that it will be further up stream, as might easily happen, by say the formation of a hook in the opposite shore line in a bend above, then, independent of the type of construction, the work installed must fail. Or should conditions develop so that the current makes a chord across the bend an especially difficult condition appears which the work may not be prepared to withstand.

These features make it necessary to revise our ordinary concept of permanence of construction and emphasize the fact that work of this class should be thought of as a continuous operation and not first construction and then an annual percentage for maintenance.

DEVELOPMENT OF METHODS OF RIVER CONTROL

The major portion of river control work on the Missouri River has been installed by the War Department in its effort to make and maintain a navigable channel. Detailed description of such work is to be found in the files and reports of the Missouri River Commission and the War Department.[2]

Retarding the current with trees or saplings, one end of each anchored to the bottom with stones and the other end kept afloat by a buoy, was about the first plan tried. The idea was borrowed from India where it was employed in rivers not subject to ice. It produced the expected result but lacked durability, principally on account of damage by ice. This method was introduced on the Missouri River 45 years ago and various modifications were tried, such as wire instead of brush and tripod instead of stone anchors. The evolution of this method resulted in a type of permeable pile dike still in use and consisting of two or three rows of piling framed together for lateral bracing and supporting a curtain of vertical poles spaced closely to more effectually retard the current. Around the piling is spread a woven brush mattress anchored down with rock. This mattress is 75 feet wide and extends downstream from the dike a distance of 50 feet.

The dike is placed so as to make a slight angle with the normal to stream flow. It is built from the bank out to the desired new bank. An objection to this form of dike is its tendency to catch drift, which, if accumulations are great enough, may lead to its destruction. To serve the purpose of creating a new bank it must be built above low water and hence the piling is subject to decay. The cost of this type of dike is in round figures about $25 per foot. While it has been used on concave banks it has not proved sufficiently substantial to justify its general use in such places, particularly where conditions are severe. Figure 3 illustrates a pile dike with drift accumulation at Leavenworth, Kans., and Figure 4 illustrates another installation with a brush mattress.

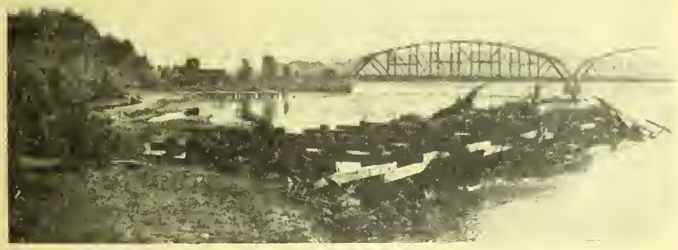

FIG. 3.—Middle dike of a series of three pile dikes protecting bridge at Leavenworth Kans. Note accumulation of drift

This type of protective work was successfully employed prior to 1923 on the convex bank opposite Waverly, Mo., for the purpose of forcing the river over against the south bluff and holding it there. At that time a relatively minor influence started the current toward the dike-protected shore. The dikes proved entirely inadequate to stop the very severe erosion and one dike was practically destroyed. Work of restoring the dikes to perform their original function is now in progress.

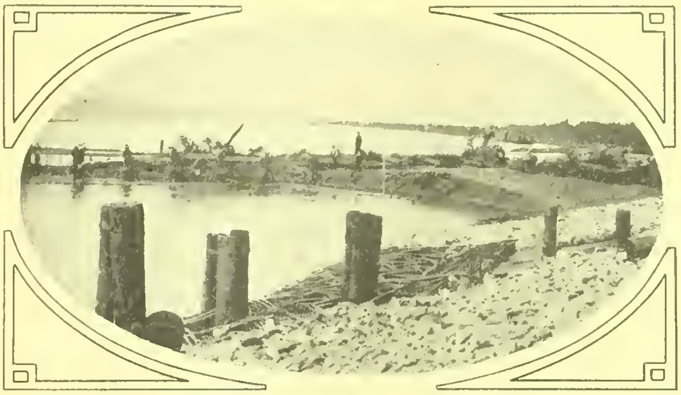

FIG. 4.—Mattress protection for new pile dike. Old dike in center of picture being replaced

BANK HEADS, LONGITUDINAL DIKES, AND ABATIS

During the early nineties experiments were made with bank heads, longitudinal dikes and a type known as the abatis. The bank heads were formed by paving the banks at intervals with mattresses of brush and stone extending well down to the possible depth of scour. They were built so as to form segments of circles in plan view. The thought was that they would so deflect the current that, if placed at frequent intervals, no erosion would occur between them. The radius was about 316 feet. This particular length was arrived at from experiments and calculations having in mind the effect of eddy currents on the downstream side. Several of these structures were built for protection of concave bends. It was found, however, that they did not prevent large bays or bights forming between them. They soon gave evidence of failure and the last was destroyed in the flood of 1903.

A case that is not unusual is that in which it is desired to correct a bight or pocket in a concave bank. Such pockets develop rapidly into large and deep bends which are more difficult to maintain and which also effect a change in the regimen of the stream further down. Longitudinal dikes have been employed to meet this condition by building the dike along the desired shore. The first longitudinal dike on the Missouri was built in 1896 just above Omaha across a bight or bay and along the line of the desired shore. It was 2,600 feet in length tied into the bank at each end, and had stem dikes placed at right angles connecting the main dike with the shore at intervals. The main dike consisted of three rows of piles with a mattress 125 feet wide, 100 feet of which lay out in front of the dike on the river bed. A second mattress was attached to the upper 750 feet of the dike supported upon a wale of the dike some 3 feet above low water. This mattress sloped from the dike outward so that when the space behind the dike had silted in the supported mattress would lie upon the newly made bank. This was said to be successful and later four more were constructed; one at Nebraska City, one at St. Joseph, one at Glasgow and one at St. Charles. The later form consisted of five rows of piling with tops 3 feet above high water. The supported mattress only was used and it sloped from the center row of piling outward and also toward the shore in long flat slopes. These installations were successful but further use of the type was not made on account of the cost which was $35 per foot at a time when common labor cost $1.10 a day.

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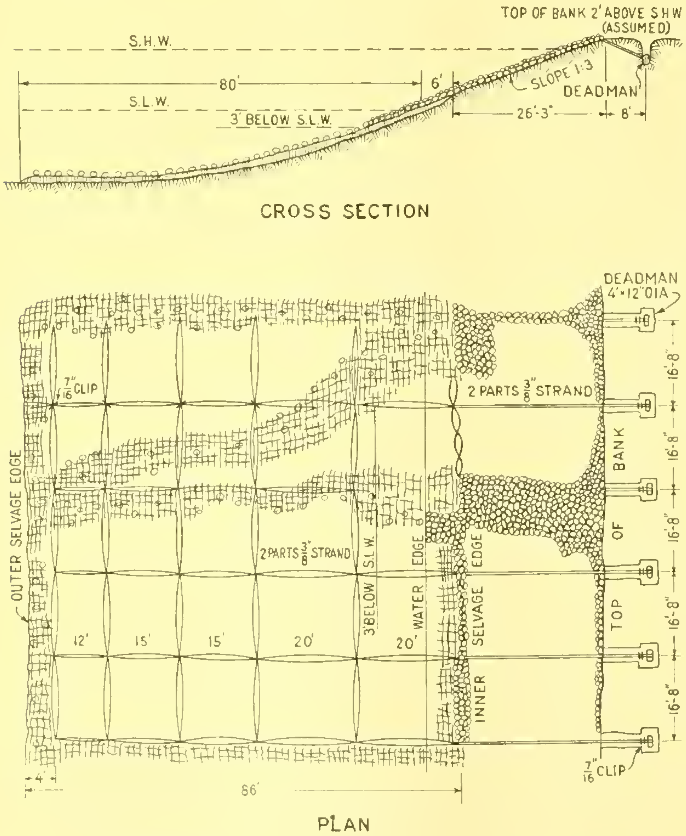
[2] A detailed description may be found in the transactions of the American Society of Civil Engineers, Vol. LIV, written by Mr. S. W. Fox, for many years principal assistant to General Suter of the Missouri River Commission.

The abatis, a type of construction named after the military device of similar shape was looked upon as a cheap form of construction to be used in closing chutes and advancing shore lines where full force of the river would not impinge against it. It consisted of triangular-shaped frames supporting longitudinal timbers which, in turn, supported poles or plank.

Bank revetment work was first used on the Missouri River in 1880. Figure 5 shows the type of wire and brush mattress finally arrived at after many years of experience. It is regarded by some as the most permanent and reliable scheme of river control. Unlike the dikes it is not subject to decay since all wood is below water; it offers no interference to drift and is not affected by ice. It accepts the river where found, hence does not of itself produce effects either favorable or unfavorable at other points. While it is ordinarily thought of as permanent its length of life is considered good if it holds for a period of twenty or twenty-five years. The revetted bank above Glasgow has failed in stretches more than a half mile long under attacks which were apparently not very severe. A revetment at Council Bluffs at what is known as the Narrows has failed but under most extreme punishment.



CROSS SECTION

PLAN

FIG. 5.—Standard form of bank revetment used on the Missouri River

Various methods of constructing and using mats have been suggested. Perhaps the most noteworthy is credited to the Wabash Railroad. This type is known as the Cunningham mat and consists of willow poles or brush laid in two directions forming a mattress about 1 foot thick which is held in position by wire fencing laid beneath and on top of the brush. The lower fence wire is tied to the upper strands at intervals of about 3 feet in each direction and enough rock fragments are laid beneath the upper layer of fencing to cause the mattress to sink. This mattress has been installed by the Wabash Railroad near Missouri City, Mo., and is said to have been successful. As the work appears to-day, much of the mattress is submerged but a larger area is spread out along the bank as safeguard against further encroachment of the stream. Figure 6 shows this construction.

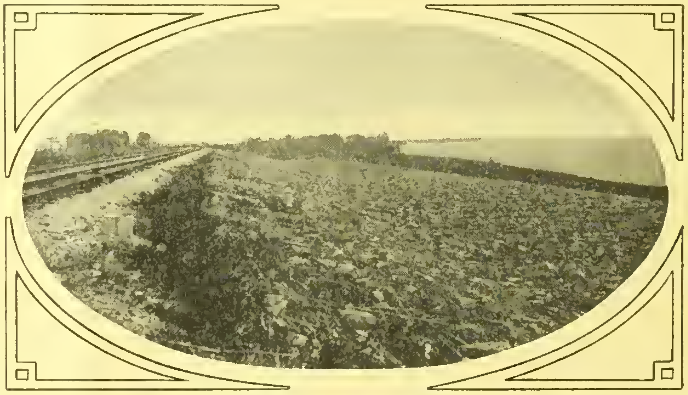


FIG. 6.—Cunningham mattress 100 feet wide protecting the tracks of the Wabash Railroad near Missouri City, Mo.

An objection to this type of construction is that the galvanized wire fencing has a limited durability, and if laid upon the bank as practiced by the Wabash Railroad, and the river does not undermine it as intended for a period of one or more years the willow brush becomes too brittle to be of much service when once submerged unless by good fortune it drops quite gently without tendency to break up. It is said to cost at the present time, including a patent royalty, approximately $20 per running foot, 100 feet wide.

The Missouri Pacific Railroad makes frequent use of a type of mattress which consists of a single layer of wire on which willow brush is laid and, over the brush and at right angles to it, heavy poles which are wired down to the lower layer of fencing. Upon this mattress, directly above the poles, is placed the rock ballast required for sinking. The cost of this type of construction is unknown but is probably not greatly different from that of the Cunningham mattress.

RETARDS ANCHORED TO SHORE

Tree retards have been used for many years by landowners in their efforts to stop the caving of banks. At first the trees both large and small, were simply thrown into the stream and anchored to dead men on the bank. Such an arrangement does give some protection but with water more than 20 feet deep it is not conceivable that the effect of floating trees so anchored, could be great. Another form of retard that has been used consists of fastening three poles approximately

16 feet long at the center so that the assembled poles form a unit as illustrated in Figure 7. Wire is then strung around the unit connecting the ends of the poles so as to form a wire entanglement. Several of these units are joined together by a cable, one end of which is anchored to a dead man on the bank, and the whole series is then rolled into the stream. These series are spaced at desired intervals, usually very close together so that the mass in the stream will form a practically continuous entanglement. This type of retard was first tried on the Kansas (Kaw) River where it is reported to be quite successful. In that stream, where installations have been seen, the entanglements partially buried themselves in the streambed, and the protruding tops caught and accumulated floating drift. Their shape and anchorage made it possible for them to hold fast and thus effect a rather substantial mass. In the Missouri River the greater depth of water is against their catching drift and unless they do in fact silt in their presence apparently becomes a cause for more active erosion due to the eddy action around individual timbers where they are in contact with the bottom of the bank.

In the Niobrara River in Holt County, Nebr., which has a wide, shallow, swift current, they proved a failure due to this cause. The poles in that installation were structural steel angles and they sank almost straight down so that their effect could not be observed within a few months.

RETARDS ANCHORED TO STREAM BED

Trees make particularly satisfactory retards if substantially anchored to the bottom because the current then forces the trees to the bottom. A concrete pile driven a few rods upstream makes an effective anchorage, the stability of which, however, is dependent upon the possibility of placing the anchorage deep enough below the stream bed to insure against displacement by the pull on the cable. While this feature is not ordinarily a factor it can be easily understood that with rock-bottom less than 70 feet deep and a depth of stream of 50 feet or more the problem is of importance. If the structure stands the first flood attack, the probability of its loss is lessened since such silting as generally occurs is an added safeguard against future dislodgment.

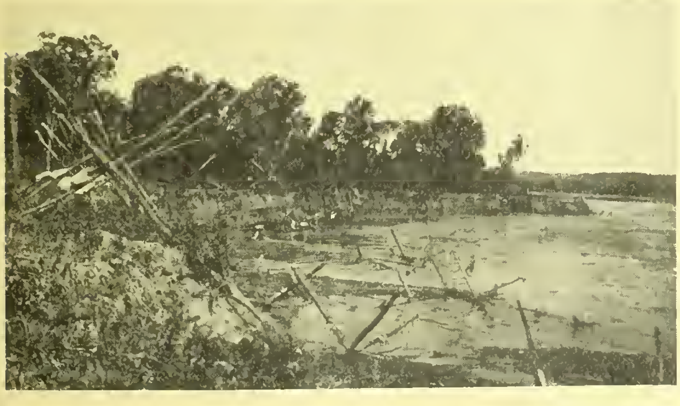


FIG. 7.—Jack-stone jetties placed in stream at point of severe attack. Those on the bank indicate the method of construction

A factor to be guarded against is the formation of eddies on the downstream side which is likely to occur if the current is especially strong or where it does not nicely parallel the shore line. Another feature which effects the permanence of this type is the breaking of cables due to the whipping of the trees. Places have been noted where the trees were still in motion two years or more after installation. It is of course a severe test of a cable to withstand the bending and twisting that goes on while the trees are continually changing their position.

In the construction of these retards the trees are piled in cordwood fashion on the bottom of the stream until they show above the water surface at ordinary stages. They are laid with butts upstream and each tree is fastened by cable to the anchor cable. The concrete pile anchors are placed approximately 35 feet apart and about 100 feet upstream from the ends of the trees. The trees near shore are piled well above the high-water stage and are anchored by cables to dead men back from the bank line. Units or piles of trees are spaced along the bank at varying distance depending mostly upon the length of the unit and upon whether the current is parallel to the shore line or approaches at an angle. The usual spacing may be taken as 500 to 1,000 feet for 150-foot units.

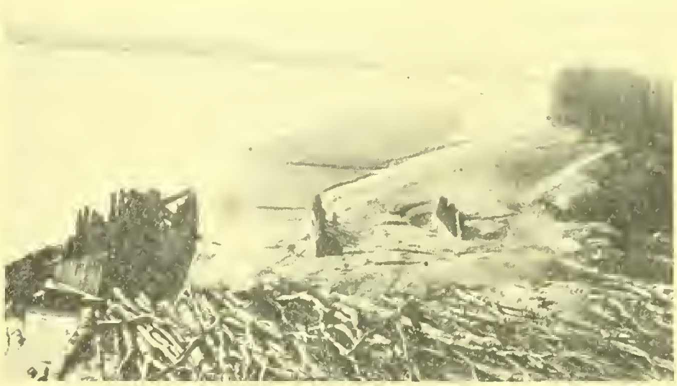


FIG. 8.—Looking downstream at tree retards from Illinois Central bridge at East Omaha. Butts of trees reversed on account of eddy action

The piling used for anchorage is about 15 feet in length and 16 inches square and is sunk under its own weight by the jetting process. The jet pipe is in the center of the pile with a device for disconnecting the hose supplying the water after the pile is in place. The trees are put in place either from a boat or from land. If a boat is used, it is anchored so that when the trees are skidded off they will take the desired position in the retard. This is done by anchoring a pulley block on shore and the pull required to skid the trees is applied from a capstan on the boat.

When trees are placed from land a temporary pile is first driven in the stream at the proposed end of the retard. To this pile is attached a pulley block and a hoisting engine placed well back on the bank furnishes the power for pulling the trees in groups of a dozen or more outward to their final position. Figure 8 shows a tree retard of this character near Omaha.

RIVER PROTECTION AT EAST OMAHA

Figure 2 shows a map of the river just above the city of Omaha. At this point the river crosses the valley from the west bluff to the east and then returns again to the west bluff and the land inclosed by this bend is called East Omaha. The area is mostly farm land but numerous houses and cottages are built in the vicinity of Carter Lake. A levee has been constructed to protect this area from overflow and in 1922 the eroding bank threatened to intercept this levee. To prevent this a series of 19 retards, covering a distance of 3½ miles of shore line, was installed. The cost per foot of retard was about $50 which amounted to $8.20 per foot of bank protected.

The general direction of the stream is straight but there exists one quite pronounced hook in the shore line and for some distance below this hook the bank is, in fact, convex. While the map does not so show, the opposite side of the river is so shallow that a bar appears during ordinary low water stage. This bar appears to force the deep water along this shore and so maintain it even where the bank is convex. Conditions at this point are not considered severe and the retards were successful in stopping erosion and have silted in irregularities as illustrated in Figure 9.

In Figure 2 it will be noted that the Illinois Central Railroad lies quite close to the river near the existing bank which was of sand. The bank at this point was first revetted but the revetment failed and the construction of tree retards was started in 1922. Construction was begun at the upstream end of the series shown at this point. After the construction of the first retard and while the others were under construction the sand bar shown on the concave bank formed and chord action of the current, though not a pronounced case, started. The current has been thrown away from the bank where it was nearest to the railroad and now impinges with considerable force against the third, fourth, and fifth retards from the lower end. These structures have held the bank but strong eddy action takes place in the bights between them. Additional trees have been thrown in these bights and anchored to dead men on shore. The trees have been whipped about violently by the whirlpools but at the time of inspection appeared to be holding on very well.

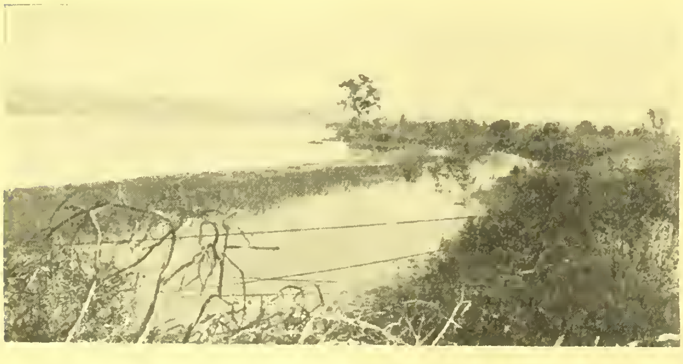


Fig. 9.—Silt deposit resulting from retard construction at East Omaha. Picture taken from tree retard a portion of which shows at lower edge of photograph

The map shows two bar outlines, the position in April, 1923, being shown by a full line and the position in October of the same year by a dotted line. At the latter date a considerable volume of water was passing close to the shore where the bar had been a few months before. In 1924 this channel again practically closed up. The stream appears to be in a state of delicate balance at the bend, so that at times the current takes the outside of the bend and at other times the short course, with resulting chord action.

At Gibson, Nebr., the Chicago, Burlington & Quincy Railroad constructed retards to protect its yards. These retards have become covered with rubbish dumped into the river and have caused the formation of a bar directly downstream. With the retard covered it seems that the current would cut away the bar. Instead of doing this, however, the bar deflects the current to the opposite bank below which is being eroded. The river at this point is quite straight and the deflection of the current by the bar is an excellent example of wedge action.

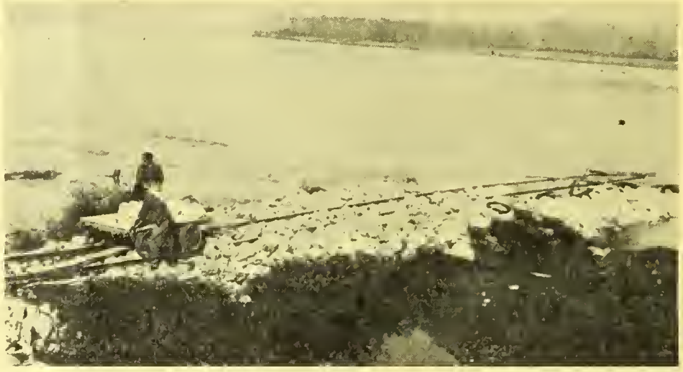


Fig. 10.—Stone and brush dike under construction showing silting that has resulted

GENERAL OBSERVATIONS ON RIVER CONTROL

An ideal plan for maintaining a river such as the Missouri in a permanently fixed channel would require holding it to a nearly uniform width at all points, shaping all bends with a maximum radius and developing as much total length as possible. This would require the protection of the outside of bends when not in contact with bluffs and the installation of structures on the inside of bends to prevent chord action. The advantage of controlling long stretches of the river as a single project is illustrated by the work of the Missouri River Commission on 45 miles of channel between Jefferson City and the mouth of the Gasconade from 1892 to 1902. This work cost about $2,500,000 or $55,000 per mile and is said to have required little maintenance since. The total amount is only about twice as great as the Burlington Railroad is said to have spent on a single bend near Folsom, Iowa.

Much of the work of river control has been done by landowners, towns, or railroads in an effort to protect their property. Generally this work has been limited to the immediate vicinity of the danger and often has been delayed until the danger has become acute. This has led to increased costs and sometimes to complete or partial failure.

For the highway-bridge engineer planning the location and protection of bridges over such streams as the Missouri River there is available the results of many years of experience in such work and he must avail himself of it if he is to construct a permanent structure.

A STUDY OF UNUSUAL EARTH ROAD CONDITIONS IN NORTHEASTERN IOWA[1]

By QUINCY C. AYRES, Associate Professor, Iowa State College

SOFT spots and mudholes which developed after grading in a number of earth roads in northeastern Iowa manifested such unusual characteristics both as to their behavior and their location that the writer was engaged by the State highway commission to conduct a field investigation of the causes of their occurrence.

In nearly all cases the soft places did not exist before the grading of the roads. Yet after grading they began to develop at the crest or along the slopes of nearly every cut despite the fact that ample precautions had been taken to insure adequate drainage in the customary manner.

Unlike the wet spots caused by side-hill seepage these failures are seasonal in character, breaking out only in the spring when frost begins to leave the ground and continuing to cause trouble until some time after the frost has disappeared. During this period they become saturated with confined water and are no more capable of supporting a load than deep beds of soft putty which they closely resemble. The occurrences are interspersed at frequent intervals between stretches of excellent roadway which bring them out in sharp contrast and render them all the more exasperating to the traveler. Certain other peculiarties have been observed, such as a tendency of the minor failures to shift position slightly from year to year, and the strange effect of rainfall which seems to improve their condition. The most plausible explanation of the latter phenomenon is to be sought in the temperature of spring rains, which are usually warm enough to melt the frost and open percolation channels.

Natural relief can not be expected until the surface evaporation becomes sufficiently rapid to dry out the top 4 or 5 inches of the road, forming a hard, tough crust. This crust, when once formed is generally thick enough to distribute loads and bridge over the soft material beneath until the following spring. At no time, however, is the crust capable of supporting heavy loads without a perceptible sag similar to that of a steel rail under the wheels of a locomotive.

Clearly these soft spots constitute a problem of a special nature which requires particular treatment for its solution. The objects of the investigation were therefore (1) to discover the cause of the trouble, (2) to determine the most feasible remedy for existing failures, and (3) to find the best way of handling future improvements so as to prevent their recurrence.

So far the occurrences have been observed only in nine counties in northeastern Iowa in the vicinity of the Mississippi River. Clayton County seems to be most seriously affected, but a number of failures have been noted in Allamakee and Dubuque Counties, and to a lesser extent the counties of Winneshiek, Fayette, Jackson, Jones, Delaware, and Clinton have also been troubled.

In Clayton County alone, the total length of the failures, in improved primary roads only, is more than 8,000 feet, and if this figure be increased in the proportion that the present improved mileage bears to the total mileage of primary and county roads in the county, it appears as probable that something like 32,000 feet of roadbed will eventually require treatment.

CONCLUSIONS AS TO THE CAUSE, PREVENTION, AND REMEDY OF THE FAILURES

As a result of the writer's investigation, in the course of which 444 test borings from 4 to 21 feet deep were made in roads in Clayton County, the following conclusions have been reached with respect to the cause, prevention, and remedy of the failures.

The soft spots have been found to occur almost exclusively at the crest or on the side slopes of cuts made in grading the roads, and the investigation indicates that they result from the exposure by the grading operations of unweathered, loessal clay which, in the unimproved road, was overlain by stable, weathered material. The water contributing to the condition is, in the main, of purely local origin. Underground sources of supply, such as springs or seepage veins, have, as a rule, been definitely eliminated.

For the curing of the existing failures the most practical remedy, in the opinion of the writer, is to remove the unstable material to a depth of at least two feet and replace it with well weathered topsoil or "black dirt," thus providing a stable crust to bridge over the unstable material during the critical period, and duplicating the condition known to exist at places where no failure has occurred. Several other remedies have been proposed, among them the covering of the affected areas with rock or sand, the addition of lime to the soil, the use of tile drains, the planting of trees which it is hoped will remove the moisture from the soil, the burning of the unstable material, the paving of soft places, and others of a less practical character. For reasons which will later be presented the writer believes the formation of a crust of weathered earth to be the most practical procedure; but it would perhaps be wise to test a number of the proposals which seem to be feasible with a view of adopting the one which proves to be the most effective and economical.

For roads to be graded in the future two methods are proposed in order to prevent the creation of the faulty condition: Either, (1) adjust the grade line to avoid cutting the hills by making heavy fills with borrowed earth; or (2) balance the cuts and fills in the usual way by making allowance for the extra depth of excavation in cuts necessary to provide for replacement of unstable material with weathered soil.

EXTENT AND PROCEDURE OF THE INVESTIGATION

The area covered and the roads studied in the writer's investigation are shown in the map, Figure 1. The evidence necessary for the determination of the cause and character of the failures was obtained by boring test holes to reveal the nature and thickness of the various strata.

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[1] This article is substantially identical with a paper presented by the author at the annual meeting of the Iowa Engineering Society at Mason City, Iowa, January 27, 1926. It is based on an investigation conducted by the writer between July and September, 1925, and upon preliminary examinations by Mark Morris and M. L. Hutton, of the Iowa Highway Commission.

The first test hole was dug at station 875+00 of primary road Nos. 10–13, about 1.5 miles southwest of McGregor (fig. 1) from which a continuous line of borings was extended to a point near Elkader at station 165+00. Two hundred and sixty borings were made in this section of graded highway, each of which penetrated the loessal deposit and extended into the till or residual soils beneath. For the most part, the holes were dug on the left shoulder of the road about 8 feet from the center line and the usual depth was from 10 to 15 feet, extreme variations being from 4 to 21 feet. The diameter of all the holes was 6 inches and the spacing in all cuts was from 50 to 100 feet or closer. A number of holes were also dug in the stable portions of the road, and a few were located in adjoining fields. At the time of making each boring depth measurements were taken and every change in color, texture or consistency of the soil was recorded.

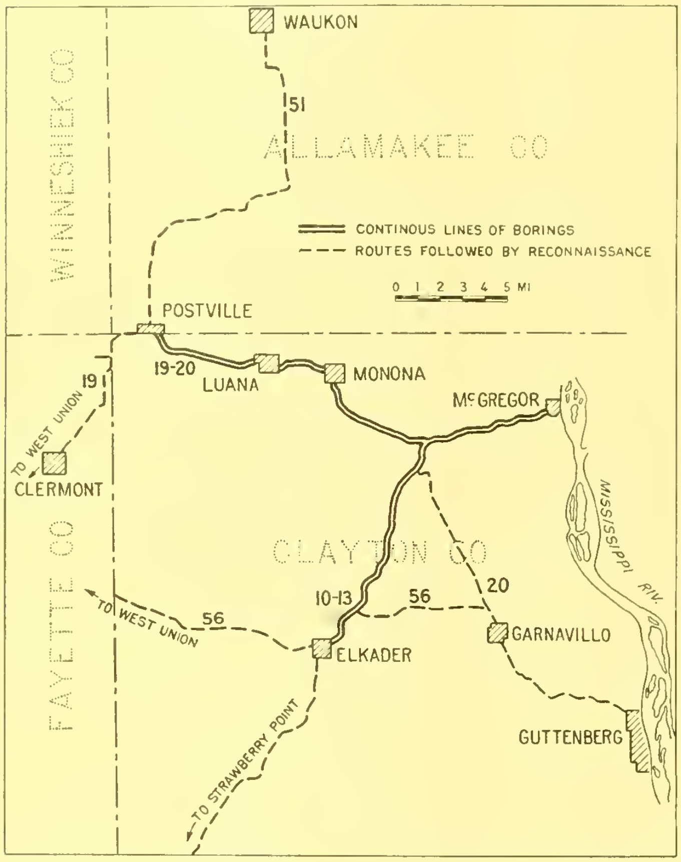


FIG. 1.—Map indicating continuous lines of borings and reconnaissance routes

The station number of each hole was accurately determined from the permanent reference hubs and a continuous line of levels was run to establish the elevation of the road surface at the holes. The elevation of the top of both banks opposite each hole was also obtained so that original surface elevations could be computed.

In this way every cut in the McGregor-Elkader road was thoroughly investigated and an attempt was made to account for those that had not caused trouble as well as for those that had. The failures of 1925 which, on this section of highway, aggregate 3,933 feet in length, were identified by laths driven in each bank opposite their extremities, and wherever such a stake was found its station number was recorded and used to locate the borings. A few unstaked failures were located from the records of County Engineer Hahn.

The next stage of the investigation consisted in boring all the proposed cuts in primary road Nos. 19–20 from Postville to Monona, at present unimproved, though the plans for such improvement are complete. One hundred and forty six holes were sunk in the manner already described except that in this road many of the borings were made along the center line in relocations. When the notes of these borings have been plotted, it should be possible to predict with a fair degree of accuracy the location of the failures that may develop after construction.

For the purpose of securing corroborative evidence, 41 holes were bored, covering the eight failures between the intersection of primary roads 10–13 and 19–20 and Monona. These failures, which have an aggregate length of 1,939 feet, were obtained from County Engineer Hahn.

In order to make certain that the soil conditions revealed by the continuous lines of boring were truly typical, the writer conducted brief reconnaissance examinations of the following roads:

Primary No. 51—Postville to Waukon.
Primary No. 19—Postville to West Union.
Primary No. 56—West Union to Elkader.
Primary Nos. 10–13—Elkader to Strawberry Point.
Primary No. 56—Elkader to Garnavillo.
Primary No. 20—Intersection of primary Nos. 10–13 to Guttenberg.

All cuts on these roads, including a number of treated and untreated failures, were closely inspected and sufficient borings were made to identify the various strata as belonging to the classification previously established from the continuous lines of borings, as shown in Figure 1. Nothing was found to controvert any of the conclusions previously indicated.

CAUSE OF FAILURE

Early in the progress of the investigation it became evident that the difference in elevation between the road grade and the original ground surface was the key to the situation and not the depth of cut below the old road, unless the two happened to coincide. Proceeding on this premise, the cause is to be sought in the geological and soil characteristics of the area affected.

Without going into an extended discussion of geologic history, this region may be said to lie in a position untouched by the last glacial invasion. Consequently it is blanketed by a layer of fine-grained, loessal clay (presumably of wind-blown origin) which covers the tough sandy clays laid down in a previous glacial epoch or residual soils composed of weathered rocks. (See fig. 2.) These underlying clays though stiff and gummy are mixed with sand and angular rock fragments which render them permeable and firm. In color they range from a deep, ox-blood red to pale tan and wherever exposed, provide an excellent roadbed.

The overlying blanket of loessal clay on the other hand, never contains any sand or rock fragments and, in general, presents opposite characteristics. Its usual thickness is from 9 to 15 feet and, though somewhat thicker on hills, as a rule it conforms closely to the residual subsurface. In its undisturbed state, three degrees of weathering are clearly marked, as illustrated in Figure 2.

First stage of weathering.—Starting at the bottom (on the residual subsurface) the first stage is noted as a pure, bright, gray clay interspersed but not mixed with thin laminated streaks of dark brown and yellow oxides and carbonates of iron (limonite striations). This clay, though fine grained, is rather stiff and requires considerable pressure to work in the hands. Its consistency in place is generally that of stiff putty, but, when dry it is powdery and fluffy, like flour. In some instances, where it overlies colored sandstone, a thin layer at the bottom of the stratum is found to be discolored to chocolate-brown, mouse-gray or ink-blue.

Second stage of weathering.—In the second stage, immediately above the first, a dull, drab gray occurs mixed with nodules of brown and yellow limonite, which has been partially oxidized. The mixing process is not so pronounced near the bottom of the layer but it gradually increases toward the top, until, in the uppermost portion, the mixture is so well mottled that its component parts are difficult to detect. In its natural condition, the clay in this stage is generally quite wet, ranging in consistency from very soft to soft putty, and occasionally stiff putty. When worked in the hands it is very soft, yielding, sticky and plastic.

Third stage of weathering.—The third and last stage represents a stratum of the same mixture in an advanced state of oxidation lying near the ground surface. It is no longer possible to discern particles of limonite nor is any of the gray clay visible. The product of complete weathering is a fine-grained clay of even texture and uniform color, ranging from a light buff-brown at the bottom of the stratum to a darker brown cast near the top. In fields, the top 18 inches or more is permeated with humus and decayed vegetation, which is responsible for the dark color and its familiar name "black dirt."

PIERCING OF WEATHER LINE CAUSES SOFT SPOTS

Weather line of the third stage.—Before considering consistency, it is well to divide this upper stratum into two parts: (1) Stable; and (2) unstable. One can never be certain just exactly where this line should be drawn, even while making borings, but perhaps three-fourths the stratum thickness below the surface would not be far from right in a majority of cases. Above this line oxidation is complete, the soil is thoroughly weathered, dark in color, crisp, firm, friable and crumbly, and is normally moist or quite dry. Below it, the color is lighter brown, the texture gummy and unyielding, and the consistency is generally that of stiff putty. Obviously, this line represents a critical elevation, since cuts that either pierce it or approach it too closely are almost certain to cause trouble. Hereafter, it will be designated and referred to as the "weather line." Above it complete weathering has occurred and conditions are stable. Below, and between it and the residual clay line, unstable conditions are found since the soil is in various transitional stages of oxidation, the latter process probably progressing in increasing degree from bottom to top.

The chemical composition of the clay in the third stage of weathering, above and below the weather line, is shown in Table 1. Practically no difference can be detected, chemically, between the stable and unstable conditions, but the colloidal content of the unweathered portion is doubtless very high.

The entire depth of the loessal deposit as well as the thickness of layers in any one of the three stages of weathering can naturally be expected to vary considerably in different localities. However, 12 feet can be said to be the most common depth with the intermediate layers spaced proportionately. On this basis, a typical section (fig. 2) may be described as follows: Total thickness of deposit, 12 feet; thickness of first stage, 4 feet; thickness of second stage, 4 feet; thickness of third stage, 4 feet. The weather line in this case would be three-fourths the thickness of the third stage, or 3 feet below the ground surface.

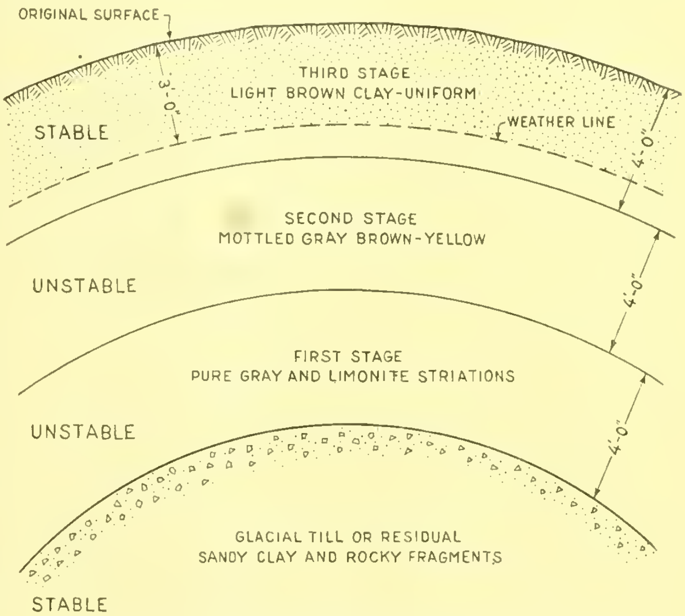


FIG. 2.—Typical diagram of three degrees of weathering of the loessal clays of Northeastern Iowa

TABLE 1.—*Chemical compositions of soil in third stage of weathering*

Substance	Chemical composition of stable material above weather line		Chemical composition of unstable material below weather line	
	Air-dry basis	Dry basis	Air-dry basis	Dry basis
	Per cent	*Per cent*	*Per cent*	*Per cent*
Silica (SiO_2)	71.10	76.41	71.80	76.28
Iron and aluminum oxides (Fe_2O_3, Al_2O_3)	17.16	18.44	16.41	17.44
Calcium oxide (CaO)	1.31	1.41	1.44	1.53
Magnesium oxide (MgO)	1.30	1.40	1.32	1.40
Alkalies and undetermined	2.18	2.34	3.15	3.35
Loss on ignition	6.95		5.88	
Total	100.00	100.00	100.00	100.00

All the foregoing discussion has been confined to soils in their original or natural state. The situation existing in failures on graded roads can not be so simply defined. Here, the normal processes of oxidation have been interfered with and the three degrees of weathering are not so clearly apparent. If the cut is deep enough to remove the third stage and sufficient time has elapsed to allow partial oxidation of the first stage, only soils in the second stage of weathering may be present. In time, these soils would no doubt gradually become further oxidized and finally pass into a stable condition, but it would be futile to hazard a guess as to the number of years required. The churning and kneading action of traffic also complicates the classification by producing an unnatural mixture at the road surface.

As long as surplus water is kept away from the upper 2 feet of this material it forms a firm roadbed with a tough, rubbery crust. Once it becomes saturated, however, (and its affinity for capillary moisture is very great) the water clings tenaciously and whatever structure it may have possessed is immediately broken up. In this condition it is extremely soft, sticky and plastic, and shifts about readily under traffic. If enough water is present it can be made to flow like thick molasses.

The only time when such a condition occurs naturally is in the spring of the year when frost is going out of the ground and large quantities of water are drawn from the wet layers beneath by capillary action. At this time, downward movement of excess water is shut off by frost and escape into side ditches is prevented by frosty, plastic shoulders. If tile drains are present, the chances are they lie in a bed of the same material which has flowed over and effectually sealed the joints. About the only way in which surplus water can be removed, then, is by evaporation which, at this season, is very slow. Relief can not come until all frost is gone and the opportunity for vertical and lateral percolation is presented.

SOURCE OF WATER

Nothing was more clearly demonstrated in the investigation than that water is not delivered under pressure from underground sources, either as springs or seepage veins. The water is purely of local origin and is held permanently at considerable depths by the peculiar capillary properties of the soil in question. In only a few instances was flowing water encountered and these were in localities remote from failures and at depths that could cause no trouble.

In most of the test holes, a layer similar to soft putty was struck at a depth of 6 to 8 feet and this extended, as a rule, to 12 feet. Below this point the material seemed to become drier. The residual sandy clays were, for the most part, relatively dry. This leads to the belief that there exists permanently, somewhere between the ground surface and the residual subsurface, a layer of saturated material which dries out slowly by percolation from below and evaporation from above. Local rains, of course, continually replenish the moisture removed in this way. The only effect of a long, hot dry summer would be to reduce the thickness of the saturated layer.

In a few places, the roots of large trees had extracted nearly all the surplus water and wherever this had occurred, borings showed the soil to be quite dry or merely damp all the way down. Other places were noted, however, where the effect of trees was not so pronounced. Strange as it may seem, the soil, in midsummer is noticeably drier in the failures than in any other part of the road. In all probability this is due to the fact that the loessal blanket is usually of minimum thickness in failures, causing a corresponding thinness in the saturated layer, which in turn produces relatively dry soil conditions because less water is held in storage.

A good idea of the changes in moisture content that occur between April and July may be had from Table 2, which gives comparative data on material taken from the same points and same depths during the months named.

MANNER OF OCCURRENCE

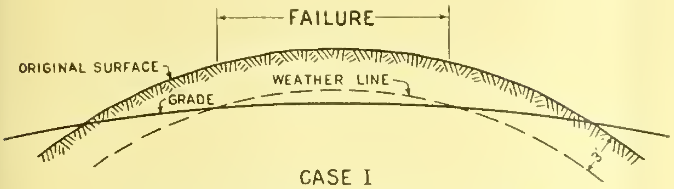
From the foregoing discussion it is plain that whenever cuts of more than 2 to 4 feet below the ground surface are made, the danger zone is generally entered and a failure is likely to occur at the crest of the puncture. (Case I, fig. 3.) If the cut is deep enough entirely to remove the loessal deposit and intersect the residual subsurface, failures commonly take place on either one or both sides of the crest. (Case II, fig. 3.) In this connection, the crest of the puncture may or may not coincide with the high point of the road grade line.

Where the road lies in a steep side-hill cut, the stable material from the high side has generally been deposited in the outer half of the roadbed and soft spots normally develop in the inner half only. (Case III, fig. 3.) Occasionally a freak formation is encountered where deep cuts, penetrating the residual subsurface, do not result in a failure on either side. (Case IV, fig. 3.)

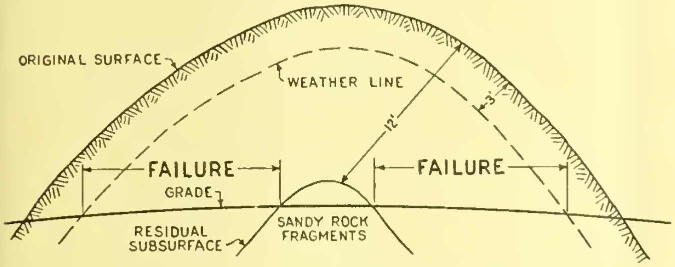
In other localities the improved roadbed was observed to be 3 feet or more below the ground surface with no failure in evidence. Such a case could usually be explained by the fact that a fill or only a very light cut had been made on the old roadbed which had previously been eroded well below the original surface. The weather line, under these circumstances, had had sufficient time to penetrate below the grade of the old road. (Case V, fig. 3.)

TABLE 2.—*Comparative moisture properties of material taken from the same points and depths within and beyond areas of failure on the Elkader-McGregor Road in April and July*

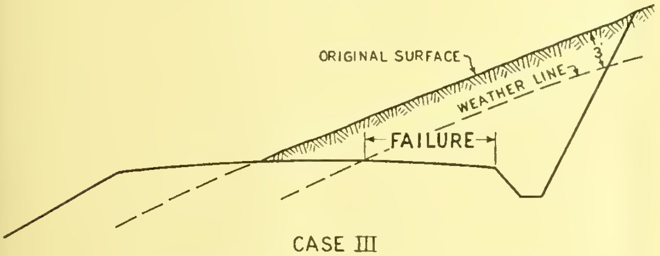
Location of test hole	Depth of strata	Depth, July sample	Description	Consistency	Capillary capacity	Moisture equivalent		Moisture content	
						April tests	July tests	April, 1925	July, 1925
	Feet	*Feet*			*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Station 556+75. Within failure.	0.5 to 4.8	2.8	Mottled gray and brown, well mixed (second stage)	Soft putty	33.8 34.5	22.1	27.6 27.6	25.9	26.4
	4.8 to 9.1	4.8	Pure gray with limonite (first stage)	do	33.7	24.8	24.8 23.8	25.4	25.0
	9.1 to 13.4	9.8	Yellow-brown sandy clay (residual)	Stiff put	28.0	18.0	23.4 20.8	18.5	21.5
	13.4 to 16	13.8	Same—slight changes in texture	Forms ball	26.2	20.7	22.2 19.2 20.8	18.3	18.2
Station 553+00. Beyond failure.	4.9 to 6.1	5.8	Light brown clay (third stage below weather line)	Soft putty	38.2	31.6	25.1 26.7 25.0	29.3	30.2
	6.1 to 8.9	7.5	Mottled gray and brown (second stage)	do	34.5	32.2	27.4 27.8	26.2	25.7
	8.9 to 17.9	10.1	Pure gray with limonite striations (first stage)	do	33.8	22.8	27.8 27.4	24.2	24.3
	17.9 to 18	--------	Sandy blue clay (residual)	Stiff putty	--------	16.8	--------	17.0	--------



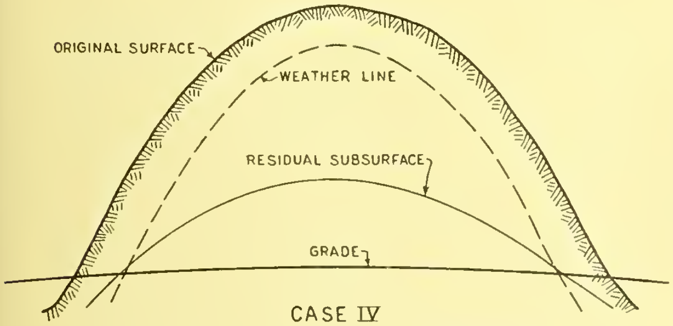
CASE I
LIGHT CUT WITH FAILURE AT CREST OF PUNCTURE
(℄ PROFILE)



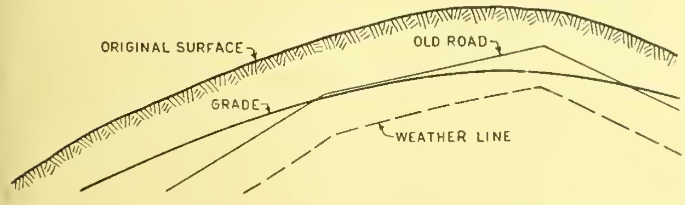
CASE II
HEAVY CUT WITH FAILURES ON EACH SIDE OF CREST
(℄ PROFILE)



CASE III
SIDE-HILL CUT WITH FAILURE AT INNER SIDE
(CROSS SECTION)



CASE IV
FREAK FORMATION IN HEAVY CUT WITH NO FAILURES
(℄ PROFILE)



CASE V
INFLUENCE OF OLD ROAD IN PREVENTING NORMAL FAILURES
(℄ PROFILE)

FIG. 3.—Classes of cuts in Clayton County roads and their effect upon the location of failure

PROPOSED TREATMENT

For improved roads.—As remedies for existing failures, many ideas have been advanced and these will be touched upon later, but it seems to the writer that the simplest, cheapest, and most practical method would be to duplicate within the failures conditions that have been found to exist outside, beyond either extremity. This means that the unstable material now present must be entirely removed to a depth of at least 2 feet and well-weathered topsoil or "black dirt" used to replace it. Every facility should be provided to allow the water from melting frost to escape to the side ditches without puddling at the surface.

For this purpose, the writer believes the excavation should extend entirely across the roadbed, should be given a slight crown, and should be paved with a thin layer of tough sod or grass roots (no long grass or weeds) before backfilling with topsoil. The sod would then act as an insulating layer separating the stable and unstable material and would also provide a permeable mat through which the water that rises from below could seep into the side ditches and flow away. Any organic matter like sod will naturally rot in time, but if air is excluded and the sod is kept moist, it is believed that its decomposition will proceed very slowly. Even after it has rotted out, many minute root cavities willl remain and provide a permeable passageway for seepage. Such use of sod can be defended on the ground that it is always removed from the base of earth dikes and dams for the reason that it does permit easy percolation of water.

The side ditches should of course be deep enough and have sufficient fall to assure the removal of seepage water as rapidly as it arrives. Any rain that falls during the critical period will be disposed of in the same way as in other parts of the road. Care should be taken to see that the topsoil extends well beyond the limits of the failure and that it feathers out into the stable portions of the road. One other point that commends itself in this regard is the fact that all materials necessary for the treatment are available at the site where needed. The essential features of the treatment by this method are shown graphically in Figure 4.

Two of the worst failures in the entire section were treated in this manner during the fall of 1925, and an inspection made during April showed the treated sections to be in good condition, although soft spots have developed as usual in other cuts and in the locations predicted.

For unimproved roads.—For roads not yet improved in this section, two methods of procedure suggest themselves. Either (1) the grade line can be adjusted to avoid cutting the hills by making heavy fills with borrowed earth, or (2) cuts and fills can be balanced in the usual way by making allowance for the extra depths of excavation in cuts necessary to provide for replacement with weathered material. The writer is inclined to favor the second method since construction can be carried on in the customary way and the treatment can be provided at small additional cost; but some situations no doubt exist where the first method would be preferable. In the long run, of course, topographical conditions would determine which is the most feasible in any given case. The first method, however, does possess a real advantage in that it is not dependent on experimental support to insure its success.

OTHER PROPOSALS

Rock treatment.—Figure 5 illustrates the method of rock treatment, suggested by Mr. Hutton, that has been tried with considerable success in a number of failures. The chief objection to its use lies in the expense of application and the fear of some engineers that it will prove to be temporary, since water has been observed oozing up between the rocks. This or any other method would doubtless be more effective and traffic conditions would be greatly improved if the unstable shoulders were entirely removed.

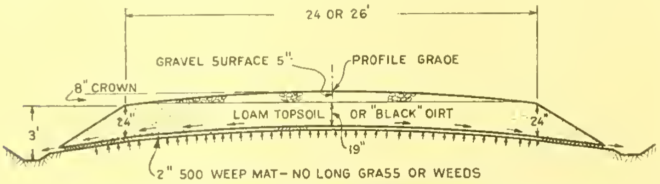


FIG. 4.—The essential features of the top soil or black dirt treatment. Any existing tile lines should be excavated and relaid in black dirt

Sand treatment.—The method of replacing the unstable material in failures with sand undoubtedly possesses considerable merit. Sand is heavier than "black dirt" and may make a firmer roadbed without hindering the passage of water. Its grains lack cohesion, however, and it would probably work its way down into the saturated clay and disappear (as gravel does at present) unless separated from the clay by a layer of boards or other impenetrable material. In exceptional places, where the grade line lies only a foot or so above the residual subsurface, sand dumped into the failure would effect a cure.

Lime.—The addition of large quantities of lime, well mixed with the clay, would be of considerable benefit in breaking up its dense and gummy structure. Experiments may show that this process progresses to a sufficient extent to cause relief. It is common practice among farmers to use lime for this purpose.

Use of tile.—There is little doubt that the benefits from tile drains as ordinarily laid in this material do not justify their cost. Even if their joints are not sealed by the plastic clay, the tile lie at such a depth as to be below the thaw line for several weeks and hence are rendered inoperative at the most critical time. After the frost has disappeared, however, they certainly would have some effect (with open joints) in reducing the amount of water present at the time of freezing in the fall.

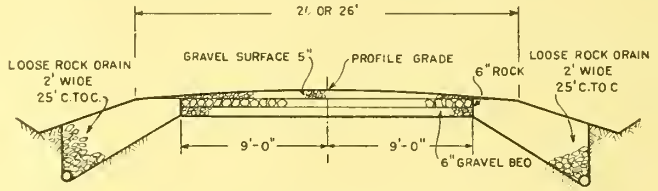
The writer can not help but believe that a line of tile under either one or both shoulders, laid in a thick bed of "black dirt," with spurs angling into the failures at frequent intervals, might effect a cure. If used in conjunction with other methods, such tile would surely serve to produce less aggravated conditions in the spring.

Trees.—All trees, and especially some varieties such as willows, have a well-known capacity to absorb large quantities of water during the growing season. Wherever a number of trees were found during the investigation close to the right of way, their effect on the moisture content of the roadbed was noticeable, and some stretches of good road can be cited that would probably have been failures without the protecting presence of trees. Where large trees exist at the site of cuts, their presence constitutes a fortunate coincidence. To plant them, however, and then await their slow development, can hardly be seriously considered as a measure of practical relief.

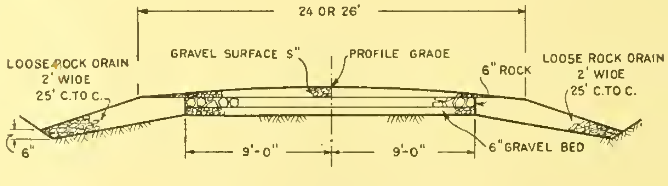
Burning.—The writer has been informed on good authority that some railroad companies, owning mileage in similar soil conditions, make a practice of burning the clay to hasten oxidation and destroy its unstable properties. Briefly, the process is said to consist in stripping off the top layer, applying a hot flame to the subgrade and then replacing the surface soil in its original position. This kind of treatment would necessitate a large plant investment, would require skilled labor to operate it, and would be expensive to maintain. It might be tried in case the less expensive and more practicable methods failed to give relief.

Short sections of pavement.—It is said to be customary in Wisconsin to cure isolated soft spots (presumably of the same nature as those in northeastern Iowa) by constructing short sections of concrete pavement to distribute traffic loads over wider areas and thus prevent failure. This practice has many features to commend it if the necessary expense can be met. Some doubt would seem to exist, however, as to whether a permanent pavement could be easily maintained with such unstable material directly beneath.

Temporary expedients.—In order to avoid closing the roads altogether for several weeks in the spring, it is common practice to bridge over the failures with heavy planks, laid directly on the yielding surface. The planks are later removed and piled at some convenient point for use the following spring. This practice requires no comment other than that it can not be condoned on any but emergency grounds.


24 OR 26'
LOOSE ROCK DRAIN 2' WIDE 25' C. TO C.
GRAVEL SURFACE 5"
PROFILE GRADE
6" ROCK
LOOSE ROCK DRAIN 2' WIDE 25' C. TO C
9'-0"
9'-0"
6" GRAVEL BED

TYPE A (ALONG EXISTING TILE DRAINS)


24 OR 26'
LOOSE ROCK DRAIN 2' WIDE 25' C. TO C.
GRAVEL SURFACE 5"
PROFILE GRADE
6" ROCK
LOOSE ROCK DRAIN 2' WIDE 25' C. TO C.
6"
9'-0"
9'-0"
6" GRAVEL BED

TYPE B (WHERE NO TILE LINES EXIST)

FIG. 5.—Rock and gravel surface treatments shown on the plans of the Iowa State Highway Commission

Another temporary expedient, has been to corduroy the wet spots with logs and long timbers placed transversely across the road. This primitive method is of historical interest since it has been used from time immemorial to bridge any and all kinds of mud holes, and, where logs can be kept permanently wet, it is entitled to some consideration. At least it can boast the merit of cheapness.

(Continued on page 66)

THE VALUE OF THE FOREMAN ON FRESNO AND WHEEL SCRAPER WORK

Reported by **ANDREW P. ANDERSON**, Highway Engineer, Bureau of Public Roads

THE value of the foreman in road grading work with fresnoes and wheel scrapers is well illustrated by data recently obtained on two jobs studied by the division of control of the Bureau of Public Roads.

The first study was a fresno job on which, at first, there was practically no effective supervision, although the work was nominally in charge of a very inefficient foreman. Later this same outfit, while operating on the same job and under practically identical conditions, was placed under the supervision of a foreman who effectively devoted his entire time to the work. The difference in the rate of operation at once became apparent and is clearly shown by the graphs in Figure 1.

Before the new foreman took hold of the job, the time taken for the performance of the several operations involved in a round trip, other than the direct haul and return amounted to 1.84 minutes, or 110 seconds, and the teams were driven at an average rate of only 179 feet per minute—a trifle more than 2 miles per hour—which is abnormally slow. When the foreman took charge, however, the time constant went down to 1.12 minutes or 67.2 seconds, while the average speed of the teams increased to 217 feet per minute or almost 2½ miles per hour. The time of performing the operations of loading, turning, and dumping, including all waits was, therefore, reduced from 1.84 minutes to 1.12 minutes, or 39 per cent, and the average operating speed of the teams was increased from 179 feet per minute to 217 feet, or 21 per cent.

The effect on the size of the average load carried to the dump was also very marked. Before the new foreman arrived the average load carried to the dump was 0.23 cubic yard. After his arrival the average load increased to 0.28 cubic yard, or over 21 per cent. For a haul of 100 feet the output of the outfit was thus increased about 83 per cent, while for a 400-foot haul, the corresponding increase in output amounted to about 64 per cent. The comparative effect of the foreman on the various operations is shown in more detail in Table 1, which represents the average results of two weeks of operation of the outfit under practically identical conditions except as to supervision.

TABLE 1.—*Stop-watch study of the fresno job before and after arrival of foreman*

Operation	Average time required		Difference	
	With foreman	Without foreman		
	Seconds	*Seconds*	*Seconds*	*Per cent*
Loading	14.4	19.3	4.9	[1] 25
Dumping and turning	17.3	20.6	3.3	[1] 16
Turning at cut	14.0	18.8	4.8	[1] 26
Throw back	3.4	5.3	1.9	[1] 36
Waiting or idle	18.0	46.0	28.0	[1] 61
Total and average	67.1	110.0	42.9	[1] 39
Average load (cubic yards)	.28	.23	.05	[2] 21
Average speed of teams (feet per minute)	217	179	38	[2] 21

[1] Decrease.

[2] Increase.

Most grading work and fresno and wheeler work in particular, consists of the consecutive performance over and over again of comparatively few and relatively simple operations. Consequently even very slight increases in the average time taken to perform each or any of these repetitive operations accumulate during the course of the day to rather surprising totals, which are clearly reflected in the reduced output. It is, therefore, not necessary for a grading outfit to cease operation for even the briefest periods in order to decrease its output by as much as 25 per cent, especially on short hauls. All that is necessary is an almost unconscious and scarcely apparent slowing down of the average rate of performance sufficient to add a few seconds to the time required for each of the several operations. The inexperienced observer would probably be unable to notice any loafing and even the workmen may honestly believe that they are just as busy and working just as hard in one case as the other. Only a stop-watch analysis or a check of the yardage moved will fully demonstrate the difference between operation directed with forethought and precision and the merely aimless hurry of undirected operation.

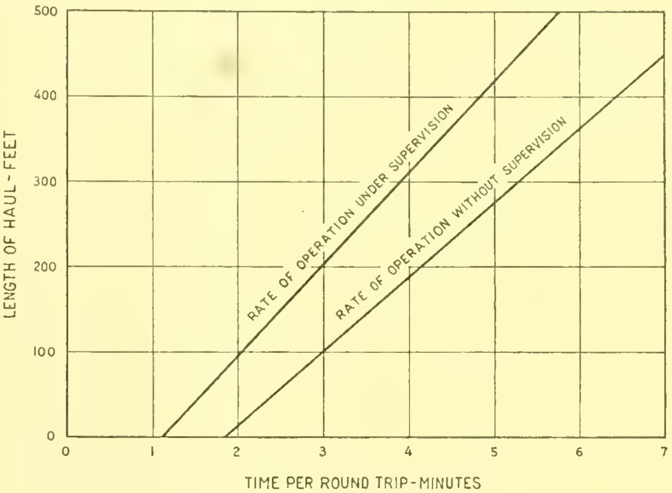


FIG. 1.—Effect of supervision on rate of operation of the fresno outfit

From Table 1 it will be seen that in this case the slowing down was of a rather aggravated form, and extended to every one of the operations, including even the speed of the teams and the size of the loads. Generally, especially during short periods of nonsupervision, the speed of the teams and the size of the load carried to the dump will be affected but little.

WHAT THE WHEELER STUDY SHOWED

The wheeler outfit presented a somewhat different set of conditions. The stock was good, the equipment first class, and the men well trained. Ordinarily the supervision was excellent and the outfit was operating at a rate well above the average. For some reason, however, it became necessary for the foreman to be absent for a period. The effect on the rate of operation was immediate and striking, as shown by Figure 2.

Under the supervision of the foreman the time constant for loading, turning, and dumping and all necessary waits was only 1.81 minutes. During his absence this time increased at once to 2.65 minutes, an increase of 46 per cent. But the average speed of the teams did not change appreciably. This was probably due, in part, to the fact that in wheeler work the pull is very light during the main hauling operation where the grade is good, as in this case, and in part to the fact that teams when in good condition and not overworked tend to maintain a fairly even pace. Furthermore, the morale of the outfit was good, so there was no conscious intent to slow up the pace or decrease production. This is further reflected in the fact that no decrease was noticeable in the average size of the load carried to the dump as determined from a count of their number and a careful cross-sectioning of the cut.

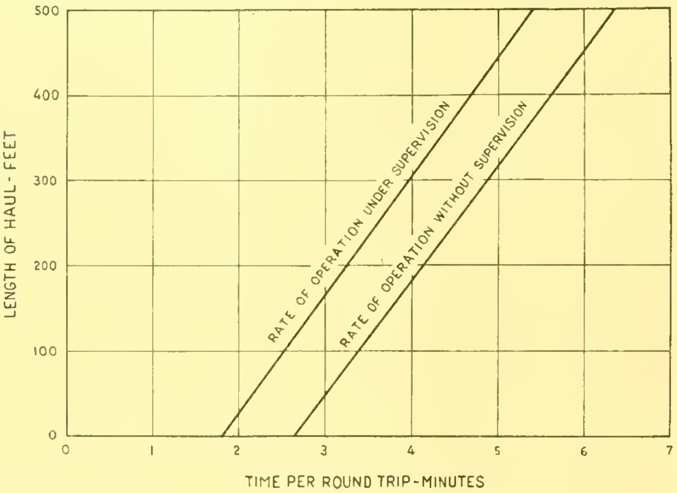


FIG. 2.—Effect of supervision on rate of operation of the wheel scraper outfit

In this case it was apparent that the slowing up was entirely unconscious; and it was manifested only in the slightly greater time required for each of the repetitive operations of loading, turning, dumping, etc., which, especially on short hauls, consume such a surprisingly large part of the working day. On a 100-foot haul the output of the outfit was decreased 34 per cent, but on a 500-foot haul, since the speed of the teams remained constant, the decrease in output, caused by the slowing up of the loading and dumping operations, was only about 16 per cent.

The influence of the foreman may be seen more clearly, perhaps, by an examination of the output per fresno and per wheeler as found in these studies. Thus, when no foreman was present on the job, the output per fresno on a 100-foot haul was 20 trips per hour, carrying a total of 4.6 cubic yards of material to the dump. As soon as the new forman had taken charge of the work this changed to 30 trips per hour carrying 8.4 cubic yards. In other words, the simple change from a careless foreman, frequently absent, to an alert man, constantly on the job, served to increase the amount of material each fresno placed in the dump when operating on a 100-foot haul by 3.8 cubic yards per hour. If figured at only 20 cents a cubic yard the value of the increased output per 10-day hour for each fresno was practically sufficient to pay the entire wages of the foreman. When the haul was 400 feet long the difference in the hourly output of each fresno amounted to 1.36 cubic yards or 13.6 cubic yards per 10-hour day per fresno. If the value to the contractor of material placed in the dump, even on this longer haul, were no greater than on the short haul, the increased output from three fresnoes would be more than sufficient to pay the foreman's wages. And since five fresnoes were usually on the job it can readily be seen that a handsome profit still remained for the contractor by virtue of the increased output the foreman brought to the job.

On the wheeler job the mere temporary absence of the foreman caused the output per wheeler to shrink from 8 to 6 cubic yards per hour on a 100-foot haul and from 3.7 cubic yards to 3.2 cubic yards per wheeler per hour on the 500-foot hauls. The foreman's absence, therefore cost the contractor 2 cubic yards per hour for each wheeler operating on the 100-foot haul and one-half cubic yard per hour for each wheeler on the 500-foot haul. Since there were usually five wheelers on the job operating on the basis of a 10-hour day it is clear that the contractor took a decided loss over and above the wages of the foreman during every hour he was absent from the job.

If these two studies are representative of average conditions there would seem to be no room for doubt that, on the ordinary grading job, a good foreman more than pays his own wages in the form of increased output and is therefore a necessary and vital part of the outfit.

(Continued from page 64)

Freak remedies.—Among the freak remedies that have been proposed may be listed, (1) chemical treatment, by which the weathering process will be completed overnight through the injection of some cheap chemical compound, and (2) the introduction of sawdust, hay, manure, or other rubbish into the failures with the hope that some magic effect will be produced that can not be clearly explained. It seems hardly necessary to add that, if any such remedies should meet with success, the good fortune will be purely accidental.

Since no theory is worth much until it has been tested by actual experience, the writer is in hearty sympathy with the plan to try a number of remedies that seem most feasible, with the view of adopting the one that proves best for general recommendation and use. He believes, however, that each experimental treatment should be carefully installed, under close supervision, so as to insure a fair trial to all.

MARYLAND TO STUDY CONCRETE CURING

Field tests to determine the relative merits of the conventional method of curing concrete with an earth covering as compared to concrete containing an admixture of calcium chloride with sodium silicate squeegeed on the surface are to be conducted by the Maryland State Roads Commission. The Bureau of Public Roads plans to have an observer present during the tests. Three sections of road each about 4,000 feet in length are to be built on the Maryland road system about 20 miles from Washington.

A unique feature of the tests is that double cylindrical molds are to be placed on the subgrade and filled and cured as a part of the pavement. Compression tests on these specimens will be made at ages of 1, 3, 7, 14, and 21 days.

COMPARISON OF TRANSVERSE AND COMPRESSIVE TESTS OF CONCRETE

By **H. F. CLEMMER**, Formerly Engineer of Materials, Illinois Department of Public Works

IT IS of primary importance in designing a concrete pavement to be able to predict within reasonable limits the actual strength of the completed slab. As Portland cement concrete has been used in the past principally where it has been subjected to compressive stresses, the compressive test has come to be general practice, and it has been carried over into the field of concrete pavement investigations, although it is a recognized fact that rigid type pavements are subjected to transverse stresses. This fact, together with the wide variation in the results of compressive tests on cores taken from pavements, has prompted general interest in the question as to whether the compressive test may be taken as a direct measure of the transverse strength of the payment.

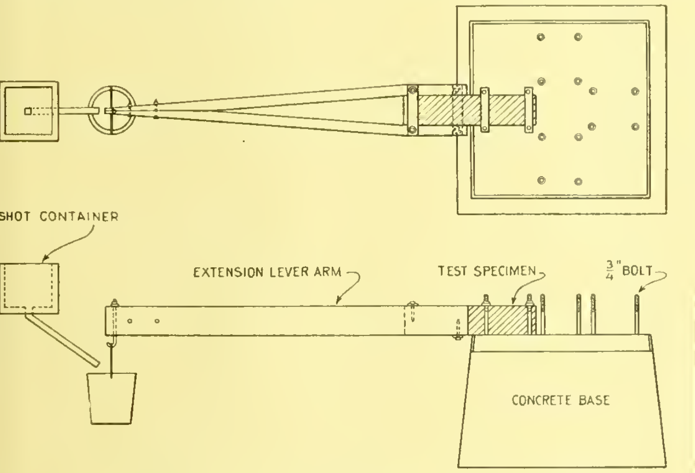


FIG. 1.—Apparatus used in making transverse tests of concrete specimens

The Illinois Department of Public Works has tested a great many cores drilled from concrete pavements and in common with the experience in other States has found a wide variation to exist in the compressive strength of the cores taken from the same job. The results have been such as to suggest that the nonuniformity may be due to the conditions surrounding the test rather than variation in the quality of the concrete. If such is the case this test indicates neither the true compressive nor the flexural strength of the slab. To throw light on this point a series of laboratory tests was conducted to determine the relation between the flexural and compressive strengths of the same concrete.

Two hundred test beams were cast, 2 from each of 100 batches and each beam was 6 by 8 by 30 inches in size. The beams were divided into three groups and one group tested at 14 days, another at 28 days and the third at 90 days. Two transverse tests were made on each beam and three compressive tests were made on cores drilled from the sections broken in the transverse test.

The transverse strengths were determined by a method developed by the Illinois laboratory which has been found to be very satisfactory. The beams are supported as cantilevers and a wooden extension arm is secured to their free ends. At its outer end the extension arm carries a bucket, as shown in Figure 1, into which shot or water is permitted to flow from another container equipped with a quick-acting valve. Uniform application of load is thus assured, and the flexural stress at the instant of breaking can be computed by taking into account the weight of the overhanging part of the specimen and that of the extension arm as well as the weight of the bucket and the shot or water it contains. The length of the specimen and the method of mounting are such as to permit more than one test to be made on each specimen; and it is particularly interesting to note that the results of tests of the same specimen rarely vary by more than a small percentage, and that exact coincidence of results is not uncommon. Figure 2 illustrates the apparatus in use.[1]

An interesting comparison of the flexural and compressive tests of identical specimens is afforded by Table 1 in which are listed the results of the two kinds of tests on 15 specimens chosen at random from the 200 beams tested. For each specimen the table shows the results of two transverse and two compressive tests and the differences between them expressed in pounds per square inch and as percentages of the minimum strength observed for each beam.

TABLE 1.—*Comparison of transverse and compressive strength of concrete specimens*

Transverse strength			Compressive strength		
Breaking strength	Difference in strength		Breaking strength	Difference in strength	
Pounds per square inch	*Pounds per square inch*	*Per cent*	*Pounds per square inch*	*Pounds per square inch*	*Per cent*
400	2	0.5	1,008	802	79.6
398			1,810		
430	17	0.4	2,000	720	56.2
447			1,280		
387	11	0.3	1,352	813	60.1
376			2,165		
415	24	5.8	3,520	1,380	64.5
439			2,140		
467	12	2.6	3,700	1,600	76.2
479			2,100		
564	13	2.4	2,610	840	47.9
551			1,770		
389	36	10.2	2,530	1,465	138.5
353			1,065		
518	11	2.0	2,520	965	62.1
529			1,555		
479	14	0.3	2,760	950	52.6
465			1,610		
441	15	3.4	2,900	1,410	94.6
456			1,400		
438	20	5.0	3,040	1,560	105.4
416			1,480		
559	7	1.3	2,150	1,310	60.9
552			3,460		
530	4	0.8	2,005	935	46.6
526			2,940		
518	23	5.0	3,400	1,680	97.7
455			1,720		
614	41	7.2	4,150	1,110	36.5
573			3,040		
Average		3.7	Average		72

The wide variation between the compressive strengths observed for the same specimen is typical of the difference observed in tests of cores drilled from the same sections of concrete pavement. That no such difference exists in the actual strength of the concrete is clearly indicated by the remarkable consistency of the transverse tests. The location and distribution of the coarse aggregate within the core as well as the nature and size of the coarse aggregate underlying the surface doubtless affect the compressive test results; and the different moduli of elasticity of the coarse aggregate and the surrounding mortar must also be considered, especially when the cores are tested in a universal testing machine which applies the load at a nonuniform rate.

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[1] This apparatus is now being used by a number of other laboratories including that of the Bureau of Public Roads.

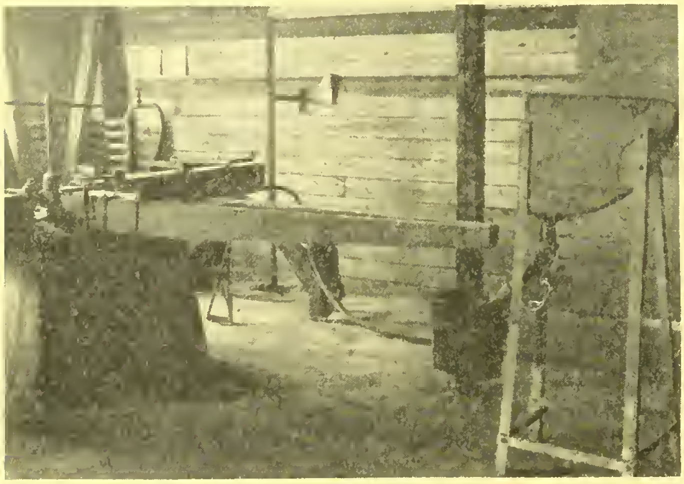

FIG. 2.—Apparatus used in determining transverse breaking strength of concrete beams

Analysis of the results of these tests suggests several questions, among which the following seem to be of sufficient importance to warrant further investigation: (1) Does the nature and strength of the outer layer or tensile fiber control the break in the transverse test or does the whole cross section at the plane of failure control it? (2) To what extent does the relative moisture content in cylinders and beams affect the respective test results? (3) To what degree does the rate of application of the load affect the test results? (4) Does the drilling of cores with the Calyx core drill cause any structural damage to the resulting core that reveals itself in the compression test?

Mainly to provide the answers to these questions the committee on tests and investigation of the American Association of State Highway Officials has planned a series of tests to be assigned to various cooperating agencies including the Illinois highway laboratory. The tests contemplated are as follows:

1. A series of tests on drilled and cast cores of concrete in which bearing areas have been carefully prepared by the arrangement of a given number of pieces of coarse aggregate. This test is suggested by G. W. Hutchinson, former testing engineer of the North Carolina Highway Commission. It has the object of determining the effect of the distribution of aggregate on compressive strength. Tests will be varied to include many combinations of aggregates. Age variation will not be an important factor.
2. Tests to determine the effect of drilling on the strength of cores.
3. Tests to determine the distribution of fiber stress in concrete beams. This may be accomplished by testing specially constructed beams having monolithic built-up layers of various types and thicknesses of concrete.
4. Development of a compression test in which uniform application of load is obtained.
5. A series of tests to determine the effect of moisture content on both compressive and transverse strength of concrete.

SLABS FOR DELAWARE RIVER BRIDGE TESTED

During the last four months the Bureau of Public Roads in cooperation with the Delaware River Bridge Joint Commission has conducted tests on concrete slabs similar in design to those used on the Delaware River bridge but smaller in size. This bridge now nearing completion is the longest suspension bridge yet constructed. The span between towers is 1,750 feet and the bridge is carried by two 30-inch cables.

The floor system consists of 6-inch concrete slabs of 1 : 1½ : 3 mix with a 2½-inch asphaltic wearing surface. The slabs are 57 feet long, by 41 feet wide and are supported by girders spaced 3 feet, 10 inches and running parallel to the long axis of the slab. The slabs are reinforced with fabricated trusses 4½ inches deep, spaced 6 inches on centers and running normal to the supporting girders and also with ½-inch round, deformed bars spaced 6 inches in the bottom and 12 inches in the top, both sets running parallel to the girders.

To check the adequacy of this design a slab similar in design and method of support but with only one-sixth the area and without the asphaltic surface was constructed at the Arlington experimental station of the bureau. Materials were the same as those used in the bridge floor. Test cylinders of the concrete showed a compressive strength of 5,000 pounds per square inch.

The first test consisted of applying a static load at the center of the slab in increments of 7,500 pounds up to a total of 30,000 pounds. Stresses in the top and bottom of the slab along both axes were measured with graphic strain-gauges. These gauges were also used to measure the stresses in the reinforcing trusses directly beneath the load. Deflection of the slab was measured at various points. As a result of this test, the entire slab was found to act as a simple plate, deflection being practically symmetrical about both axes. The maximum stress found in the trusses was 11,000 pounds per square inch. Maximum compression in the concrete was 580 pounds per square inch along the axis normal to the supporting beams and 1,200 pounds per square inch along the other axis. The deflection under the 30,000 pound load was 0.1 inch and no permanent set was found.

Impact tests were next made at the quarter point of the slab and centrally between supporting beams. The impact machine was adjusted to represent a wheel load of 15,000 pounds, the unsprung weight being 2,060 pounds, and this load was dropped one-half inch. A series of 3,000 blows was delivered, resulting in the formation of four hair cracks under the load extending outward for a distance of 14 to 18 inches. These cracks were 6 to 8 inches in length at 300 blows and showed no increase after 1,000 blows. The maximum deflection was 0.120 inch and no permanent set was measurable.

The impact machine was then moved to a new point and adjusted to give a drop of one inch without change in loading. The impact pressure developed was 18,540 pounds which is approximately equivalent to that of a 7½-ton truck with maximum overloading. After 3,000 blows no change was noted other than the formation of six hair cracks, under the load, 16 to 18 inches long, although the slab was loosened on the supporting beams and moved diagonally about two inches.

From these tests it is concluded that the design is sufficient for any loading likely to come upon the bridge. Additional tests are now being made to secure information for use in designing this type of slab.

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
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*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
No. 1259. Standard Specifications for Steel Highway Bridges; adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94. TNT as a Blasting Explosive.
331. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60. Federal Legislation Providing for Federal Aid in Highway Construction.

FARMERS' BULLETINS

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*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

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*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.
914. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

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Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

APRIL 30, 1926

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 11,167,362.27	$ 5,367,803.24	622.3	$ 5,846,496.88	$ 2,768,529.70	263.7	$ 133,112.16	$ 66,666.08	0.8	$ 3,293,368.12	Alabama
Arizona	9,680,133.43	5,016,119.94	613.8	1,230,100.53	748,233.76	102.3	1,067,930.65	716,949.79	78.8	637,790.19	369,162.68	28.4	2,767,782.63	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	4,414,693.51	1,972,083.73	237.0	5,176,465.22	2,488,384.49	331.6	536,670.32	236,150.00	37.4	1,528,994.06	Arkansas
California	22,346,176.99	10,719,249.61	894.8	4,135,497.87	1,896,396.86	138.8	11,389,368.70	5,669,451.89	326.2				3,798,717.65	California
Colorado	11,876,703.94	6,067,814.34	661.2	1,186,883.79	616,367.01	66.6	4,530,171.83	2,314,740.49	239.7	683,307.79	276,461.34	20.7	3,061,438.62	Colorado
Connecticut	4,558,630.29	1,819,368.66	101.6	866,927.90	281,217.14	15.5	1,788,444.35	569,254.68	27.1	1,063,817.89	309,237.47	14.3	1,364,603.06	Connecticut
Delaware	4,281,659.81	1,496,190.66	107.1	636,492.48	286,474.96	17.2	791,364.06	316,854.76	17.8	906,853.78	196,263.44	22.7	180,274.21	Delaware
Florida	2,959,273.72	1,406,487.97	96.3				10,476,626.23	4,978,155.14	309.2				1,701,310.89	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	4,087,584.44	1,989,350.88	243.8	11,293,499.44	6,672,699.50	692.0	2,190,368.48	1,069,889.63	60.8	393,636.63	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	1,471,406.96	923,019.03	113.8	2,369,733.91	1,484,469.49	121.1	1,094,604.27	624,863.37	84.6	711,942.86	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	3,367,972.82	1,623,785.54	113.1	6,990,923.37	2,892,046.86	204.5				6,676,290.32	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	2,637,704.28	1,229,628.96	87.6	16,674,122.40	7,796,493.09	450.3	527,415.47	261,266.46	13.2	2,364,510.82	Indiana
Iowa	27,272,286.21	11,107,492.99	1,996.9	722,482.52	320,365.06	36.6	9,087,872.52	4,103,461.89	667.1	3,367,948.70	1,649,290.69	256.7	2,304,962.37	Iowa
Kansas	26,399,696.77	9,766,273.32	831.4	6,381,456.99	2,812,491.49	324.4	9,989,632.53	4,022,926.80	536.9	2,213,880.93	709,606.67	147.0	2,164,113.72	Kansas
Kentucky	14,832,324.28	6,206,994.69	584.9	4,492,826.73	1,670,347.39	116.6	6,600,186.13	3,122,726.52	322.7	319,991.08	143,098.54	3.0	2,070,642.96	Kentucky
Louisiana	11,939,424.97	6,279,870.86	927.6	1,740,934.40	792,419.06	114.7	3,447,327.14	1,673,265.76	162.9				1,526,862.33	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	673,271.45	284,637.06	22.2	1,809,496.61	706,814.24	67.6	92,205.65	46,102.82	4.7	1,619,403.55	Maine
Maryland	8,132,506.90	3,849,383.16	294.4	2,792,436.20	1,263,508.07	128.9	169,996.00	84,997.50	10.8	1,408,591.11	661,317.24	70.9	65,761.04	Maryland
Massachusetts	14,047,666.22	6,467,661.28	300.6	3,618,699.61	1,006,264.48	61.8	3,486,954.61	967,291.35	48.2	505,268.22	146,144.20	7.6	2,532,364.69	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,613,563.66	4,437,617.17	346.6	9,009,666.64	4,059,652.31	219.1	191,176.00	83,048.13	6.3	4,433,830.48	Michigan
Minnesota	30,416,686.89	12,738,642.04	2,721.2	6,271,152.99	2,284,974.62	397.0	8,504,674.84	2,650,600.00	656.3	1,399,085.94	276,900.00	60.0	1,630,663.44	Minnesota
Mississippi	10,292,286.79	4,988,702.73	803.4	3,046,890.27	1,621,945.91	226.4	8,025,201.25	3,981,605.94	417.2	1,344,598.93	580,446.41	39.6	1,065,317.01	Mississippi
Missouri	17,368,156.67	8,219,411.43	1,118.9	9,012,250.86	4,338,296.13	326.6	21,126,049.48	8,446,116.99	692.3	1,378,318.46	610,483.74	52.9	1,172,128.71	Missouri
Montana	10,156,600.41	5,317,523.16	921.6	1,244,383.40	1,015,942.74	133.3	1,389,699.51	936,734.74	107.3	946,723.12	613,821.09	110.2	5,640,863.28	Montana
Nebraska	9,306,374.36	4,389,523.50	1,670.6	1,968,824.26	956,092.64	167.3	11,339,124.08	5,616,483.49	1,144.8	2,073,991.43	1,032,612.40	207.6	2,640,522.97	Nebraska
Nevada	4,917,466.68	3,088,299.78	367.3	2,207,013.84	1,727,392.04	167.8	3,677,844.16	3,026,916.07	342.2				952,608.11	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	826,870.74	391,223.20	29.5	682,611.65	274,126.52	17.7	46,925.79	22,962.89	1.4	494,953.62	New Hampshire
New Jersey	11,961,357.46	3,820,679.99	219.1	4,110,683.94	1,191,983.72	65.6	6,193,163.14	2,434,752.44	26.0	420,870.40	66,450.00	4.4	953,653.86	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	3,568,872.54	2,326,079.86	329.6	1,466,200.02	966,613.62	116.4	74,555.37	46,757.90	6.4	2,718,864.01	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	10,721,251.06	4,261,605.31	273.5	33,121,607.68	9,548,602.24	602.8	6,689,573.00	1,244,765.00	83.3	6,761,145.92	New York
North Carolina	21,014,450.41	8,746,454.69	1,119.8	3,923,647.63	1,699,863.21	93.9	8,784,054.64	3,731,110.13	210.0	1,644,639.01	822,319.50	44.0	817,468.67	North Carolina
North Dakota	10,829,263.82	6,268,930.47	1,917.6	1,437,900.21	739,866.63	276.6	3,572,688.81	1,768,351.71	446.7	2,596,504.84	1,434,751.66	376.0	1,536,769.64	North Dakota
Ohio	41,672,252.81	16,244,993.93	1,191.1	6,069,391.44	2,107,214.63	171.8	8,531,417.31	3,394,848.69	272.2	4,161,653.81	1,383,537.13	108.9	3,601,201.62	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	6,514,676.70	2,661,712.80	226.6	4,580,792.94	2,182,243.50	190.7	634,804.62	307,281.98	31.3	1,335,668.38	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,326,783.62	1,256,646.21	111.3	2,290,319.28	1,378,618.00	119.0	1,484,192.23	714,382.05	65.7	387,337.11	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	15,769,432.05	4,561,888.14	286.9	27,050,703.72	7,588,139.38	546.2	2,862,755.50	930,606.84	61.5	2,036,122.67	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,360,119.89	439,140.97	21.9	1,531,802.80	427,155.00	28.6	45,705.55	13,050.00	0.9	668,534.94	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,236.9	3,116,050.26	1,313,931.11	169.4	6,394,991.78	2,856,526.84	287.3	663,763.43	306,165.95	51.3	203,632.56	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1,447.9	4,727,532.26	2,292,649.22	649.0	3,241,938.78	1,538,501.55	501.9	89,856.30	44,928.16	18.0	1,300,832.08	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	7,522,750.94	3,388,679.66	269.4	6,901,688.34	3,220,489.13	261.7	1,319,807.48	539,903.06	19.4	1,399,439.39	Tennessee
Texas	54,120,970.83	21,067,940.12	3,907.1	13,636,268.72	5,754,767.46	827.9	18,908,133.36	8,449,040.77	1,048.8	4,012,119.36	1,794,109.09	113.1	3,550,573.57	Texas
Utah	6,269,169.41	3,818,836.91	423.1	1,211,294.46	764,474.36	63.1	2,488,054.96	1,789,788.36	214.3				1,445,679.38	Utah
Vermont	3,016,174.51	1,452,884.46	107.8	1,226,868.13	664,805.06	26.7	1,024,826.46	423,906.57	23.2	483,741.14	169,091.25	6.7	657,810.67	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,316,348.84	3,848,702.89	303.6	6,929,855.37	3,031,274.42	209.0	692,816.29	288,497.54	17.8	61,040.86	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,691,201.52	1,607,897.59	137.2	2,526,208.66	1,232,400.00	27.6	793,644.62	424,000.00	10.2	764,266.64	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	1,639,091.90	673,672.94	67.1	6,593,959.67	2,223,538.47	141.3	1,016,249.04	427,341.73	42.0	797,764.53	West Virginia
Wisconsin	21,807,140.91	8,919,640.82	1,461.7	2,994,612.24	1,446,736.86	137.4	4,194,026.92	2,096,298.69	204.3	2,088,894.83	1,028,337.60	110.0	3,947,802.34	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	2,049,669.09	1,258,188.38	160.8	2,421,548.69	1,525,794.02	160.8	184,915.70	118,584.00	16.0	924,610.93	Wyoming
Hawaii							1,050,897.93	312,635.18	16.9				787,517.82	Hawaii
TOTALS	740,140,790.82	326,654,346.00	41,898.3	193,325,711.19	85,704,371.13	8,962.6	334,328,211.86	143,340,261.69	13,808.6	53,699,718.00	21,990,655.69	2,423.6	94,685,475.49	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 71,916,541.14 Federal aid $ 32,193,281.99 Miles 2,967.6

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 4 ▽ JUNE, 1926

THE ROUGH STONE BLOCK TEST HIGHWAY USED IN THE IMPACT TESTS

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1926

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 7, NO. 4 JUNE, 1926 H. S. FAIRBANK, Editor

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MOTOR TRUCK IMPACT AS AFFECTED BY TIRES, OTHER TRUCK FACTORS, AND ROAD ROUGHNESS

Report of Cooperative Tests by the Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America

Reported by **JAMES A. BUCHANAN**, Bureau of Public Roads, and **J. W. REID**, Rubber Association of America

SINCE 1923 there has been in progress at the experimental station of the Bureau of Public Roads at Arlington, Va., a series of motor-truck impact tests designed especially to develop information with respect to the influence of various types of tires, certain elements of vehicular design and equipment, the loading, capacity, and speed of the vehicles, and the roughness of the road surface. After ascertaining the magnitude of the impact under the various conditions created it is the further object of the tests to determine the stresses induced in the road surface. Up to this time, however, efforts have been directed principally toward the attainment of the first objective, and it is to this phase of the investigation that the present report is confined.

The entire investigation is being conducted cooperatively by the Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America, under the direction of a joint committee representing the three organizations.

Owing to the necessity of developing an accelerometer adapted to the conditions of the tests, accumulation of data bearing upon the specific objects contemplated could not be begun until 1924. Since then the tests originally scheduled have been completed and the data are available in standardized final form.

The results of the tests reported below were obtained with equipment specified by the cooperative committee. The tires used were standard equipment at the time of testing, and the results obtained apply specifically to these tires. They do not necessarily apply to tires that have been redesigned during or subsequent to the tests. This is particularly the case with reference to tires which have been specially designed for use in dual mountings, the profile heights being appreciably affected.

In applying the test data it should be remembered that an impact force, expressed in pound units, does not necessarily have the same effect upon materials as a static force of the same numerical value in pounds.

The term "cushioning effect" as used refers only to the vertical reaction between road and wheel, and is not to be confused with the popular term "riding quality," although it is somewhat akin to it. It should be borne in mind also that this research has been concerned with only one phase of certain economic problems, full consideration of which should give due weight to various other highway transportation factors relevant to the subject.

IMPORTANT CONCLUSIONS TO DATE

From the data at present available—and it should be borne in mind that they were obtained from the standard equipment used at the time of the tests—certain preliminary deductions may be drawn, namely:

1. Maximum impact forces obtained with motor truck tires in service can be measured with an accuracy sufficient for the needs of this investigation.
2. As static load increases, road impact reaction increases.
3. As static load increases, the ratio of road impact reaction to static load decreases.
4. Thickness and narrowness of tread rubber are desirable in reducing road impact reaction.
5. Increasing the thickness or profile height of rubber has a very marked effect in reducing road impact reaction in both single and dual mountings.
6. In the tire equipments tested, all of which were standard at the time of the tests, dual mounting caused heavier impact forces than the corresponding single mounting of the same total load-carrying capacity. (This was determined on a pneumatic-tired, 2-ton truck and a solid-tired, 5-ton truck.)
7. Appreciable variation of cross-sectional rubber, or breaks in its continuity, cause heavy repeated impacts to be delivered to the road.
8. Dual-mounted tires should always be mounted with the tread design staggered.

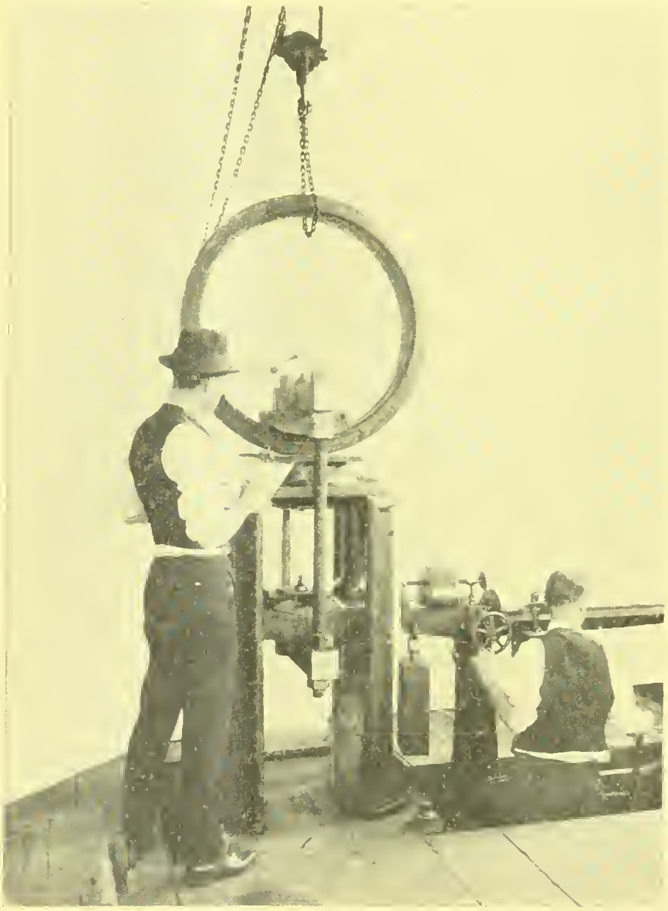
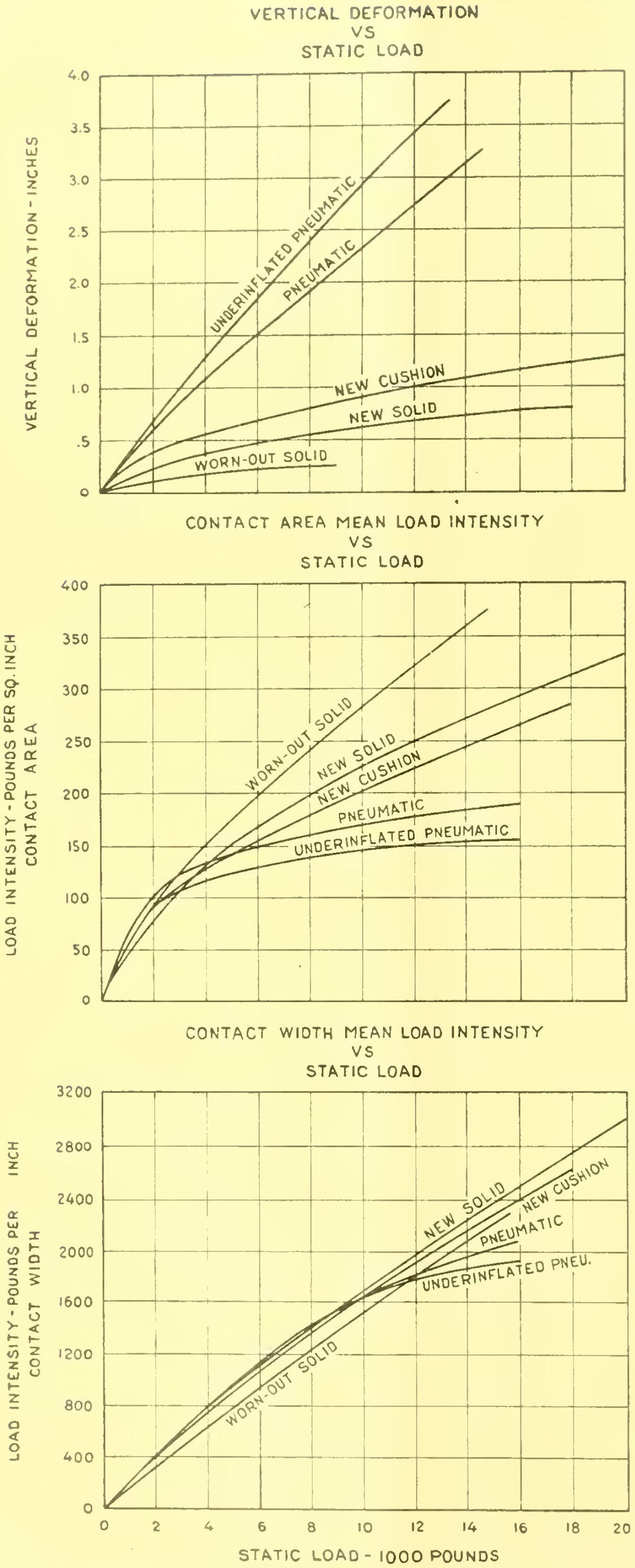


FIG. 1.—Method of mounting tire for static test

PROCEDURE OF THE TESTS

The tests of tire cushioning qualities are divided into two general groups, the static tests and measurements, and the impact tests. It is expected that, in most cases, a general relation will be found between these two test methods which will shorten and standardize


VERTICAL DEFORMATION VS STATIC LOAD

CONTACT AREA MEAN LOAD INTENSITY VS STATIC LOAD

CONTACT WIDTH MEAN LOAD INTENSITY VS STATIC LOAD

Fig. 2.—Typical curves showing tire action under static load

future test procedure. It may be that a relation of load versus work will be found which will yield a useful criterion in the correlation of the static and the impact tests.

Static tests.—In the static tests the tires were placed in a testing machine and vertical deflections were recorded as the load was slowly applied, from which data load *vs.* deformation curves were drawn. The areas of tire contact were taken at three loads on each tire, and reproductions were made of the impressions. From these the area, width, and length of contact were obtained at the three loads. The hardness of the tread rubber was measured by a Shore durometer. In addition the weights, dimensions, and trade names of the tires were recorded.

The method of mounting tires in the testing machine is shown in Figure 1; and typical curves indicating tire action under static loads are shown in Figure 2. These curves show the effect of load on vertical deformation, the mean intensity of load per unit of contact area, and the mean load intensity per unit of contact width. Figure 3 shows typical tire cross sections.

Impact tests.—In the impact tests the total vertical road reaction is determined by a specially constructed accelerometer mounted on the test trucks. This accelerometer was designed to measure the maximum vertical accelerations of the truck wheel and simultaneously to record the proportional deflections of the corresponding truck spring. The forces are computed by formula and may be resolved into sprung and unsprung components. The sprung component is determined by measuring the change of truck spring deflection from its static position at a given load to its position at the instant of impact. By referring to the proper portion of a previously prepared calibration curve for the truck spring, the change in spring pressure from that at static load is obtained. The unsprung component is determined by measuring the acceleration or deceleration of the truck wheel, adding thereto the acceleration of gravity and multiplying the sum by the known mass of the unsprung parts (i. e., the parts below and not supported by the truck springs).

The corresponding total force or road reaction is computed according to the formula:

$$F = m\,(a+g) + P,$$

wherein F = vertical road reaction in pounds,
m = mass of unsprung truck parts in poundals,
a = acceleration of unsprung truck parts in feet per second per second,
g = acceleration of gravity (32.2 feet per second per second),
P = truck spring pressure in pounds at instant of impact.

THE ACCELEROMETER

The accelerometer is mounted to follow faithfully the vertical movement of the right rear wheel of the truck by means of a tight, self-aligning bearing, and the instrument slides on guides on the truck body. The sensitive element is composed of a calibrated coil spring supporting a weight of known mass. A rod fastened to this weight acts as a guide and stylus holder. The stylus records the relative movement of the weight on sensitized paper which is rolled on a drum driven by the truck wheel, the paper moving approxi-

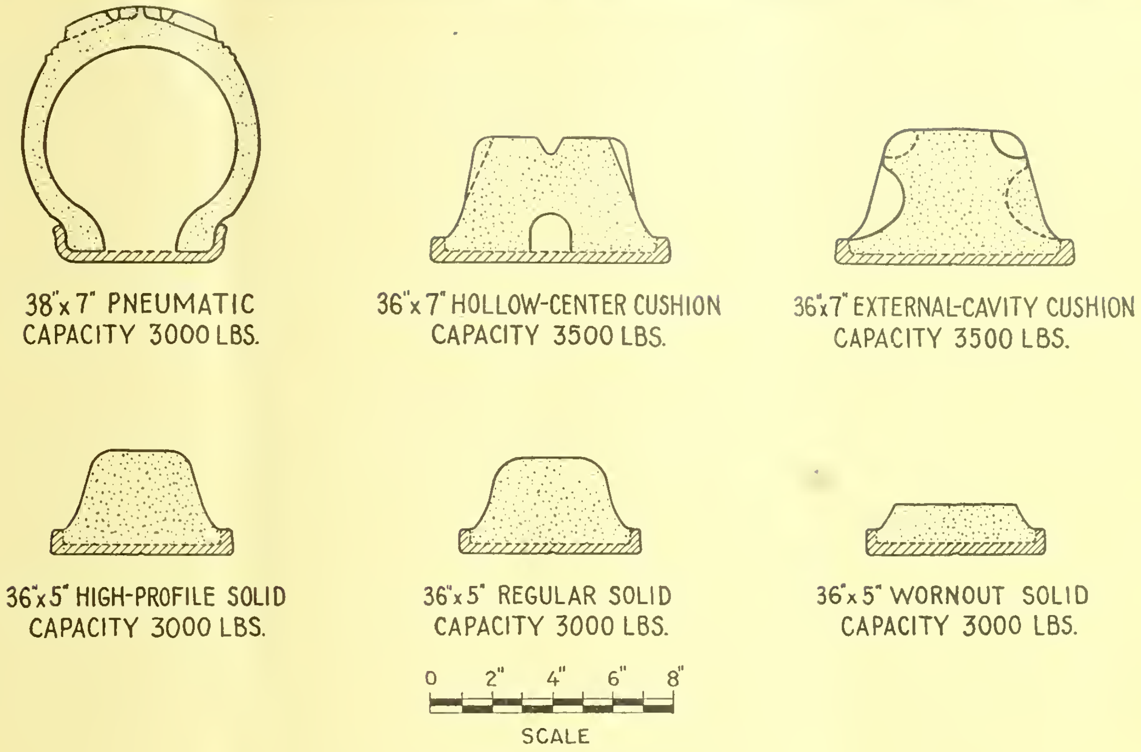


Fig. 3.—Typical tire cross sections

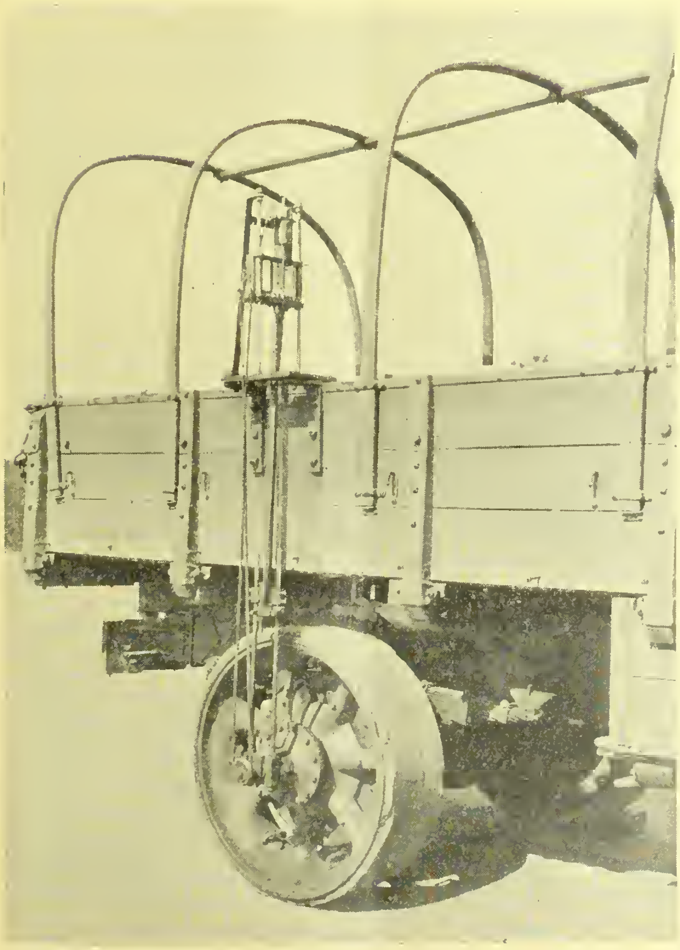


Fig. 4.—General view of instrument mounted on test truck

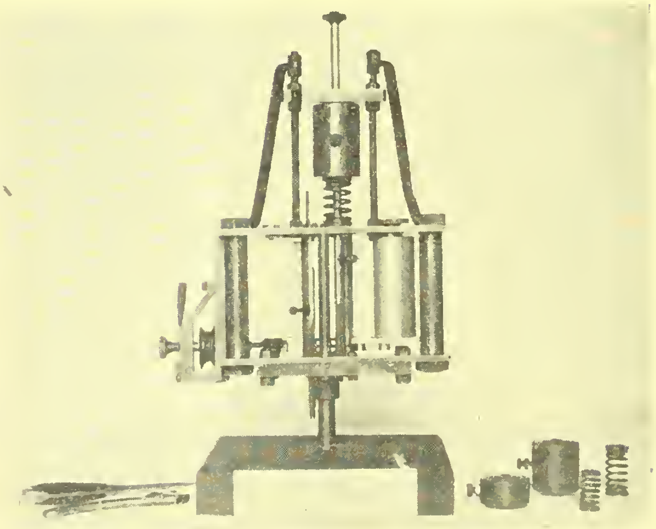


Fig. 5.—Detail view of instrument

mately 1 inch for each 45 feet of truck travel. A backstop is arranged above the accelerometer weight to prevent the upward movement of the weight beyond the position it assumes when the instrument is at rest and there is no initial compression on the spring other than that due to the weight itself. Figure 4 shows the complete apparatus installed on a truck. Figure 5 is a close-up view of the instrument proper, and the essential elements of the instrument are shown diagrammatically in Figure 6. Typical accelerometer records and the process of converting them into terms of force are shown for artificial obstructions in Figure 7 and for an actual road surface in Figure 8.

In Figure 7 the spring reading is recorded at zero for the condition of shock at each of the narrow obstructions because, as indicated by the instrument record, the truck spring is in substantial equilibrium immediately preceding the approach to each of these narrow obstructions. This is due to the fact that the obstructions have been placed on a smooth road. At the instant of impact with the obstruction there is an added reaction between the truck wheel and the pavement which may amount to many times the static loading. This is due to the acceleration of the unsprung weight although the sprung weight is still at its zero position. Strictly speaking, however, at the instant of maximum upward acceleration of the truck wheel, the truck spring has been compressed an amount equal to a small percentage of the height of the obstruction. For pneumatic tires this compression is about 10 per cent, for cushion tires 15 per cent, and for solid tires 20 per cent of the height of the obstruction. The spring record curve has sharper peaks downward than upward because the downward peaks represent the stopping of the wheel by the pavement, whereas the upward peaks represent the upward motion of the wheel against the resistance of the truck spring.

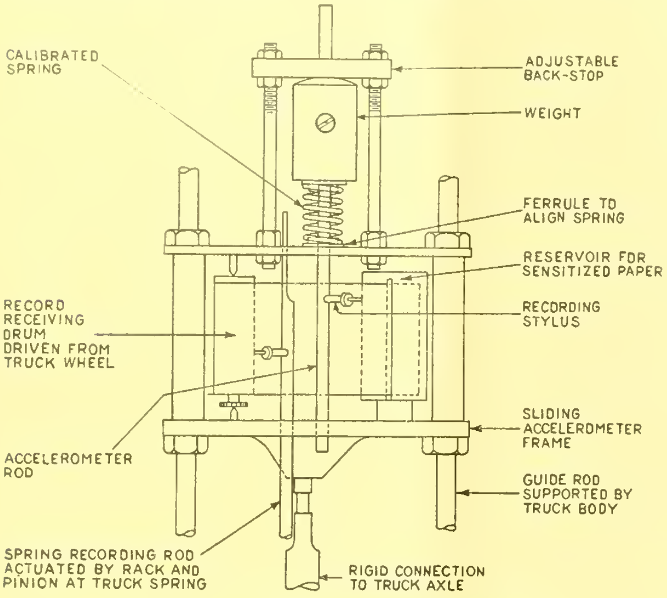
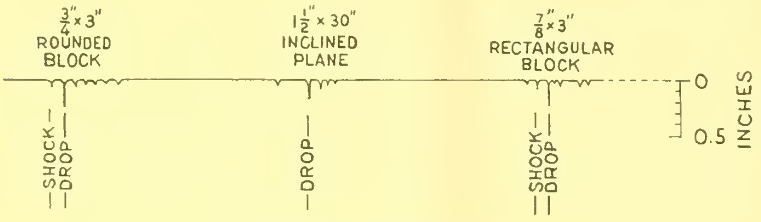

CALIBRATED SPRING
ADJUSTABLE BACK-STOP
WEIGHT
FERRULE TO ALIGN SPRING
RESERVOIR FOR SENSITIZED PAPER
RECORD RECEIVING DRUM DRIVEN FROM TRUCK WHEEL
RECORDING STYLUS
SLIDING ACCELEROMETER FRAME
ACCELEROMETER ROD
GUIDE ROD SUPPORTED BY TRUCK BODY
SPRING RECORDING ROD ACTUATED BY RACK AND PINION AT TRUCK SPRING
RIGID CONNECTION TO TRUCK AXLE

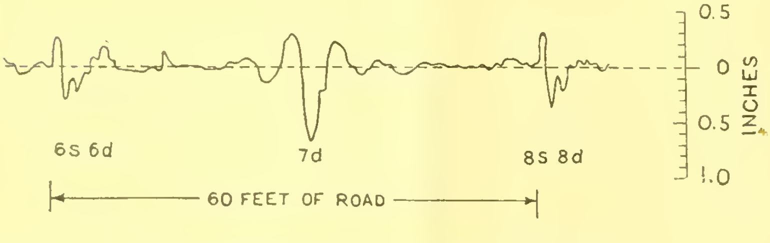
FIG. 6.—Diagram of the coil spring accelerometer

The characteristics of the accelerometer are such that the displacement of its weight as recorded on the sensitized paper is proportional to the acceleration of the truck wheel. The calibration factor depends on the characteristics of the accelerometer spring-weight combination and varies with the characteristics of the acceleration impressed on the accelerometer by the truck wheel. The accelerometer spring-weight characteristics are accurately determined and that combination is selected which is best suited to the accelerations to be measured. The characteristics of the impressed acceleration are influenced by the natural characteristics of the tire equipment and the unsprung truck weight in the case of the wheel dropping on a plane surface. In the case where the wheel passes over a relatively narrow obstruction at comparatively high speed, there may be an added correction dependent on the duration of this condition of shock.



ACCELEROMETER RECORD

The deviation of the accelerometer record from a straight line is a measure of the acceleration of the unsprung parts. This acceleration multiplied by the mass of the unsprung parts gives the force of the blow struck by the unsprung parts.



CORRESPONDING TRUCK SPRING RECORD

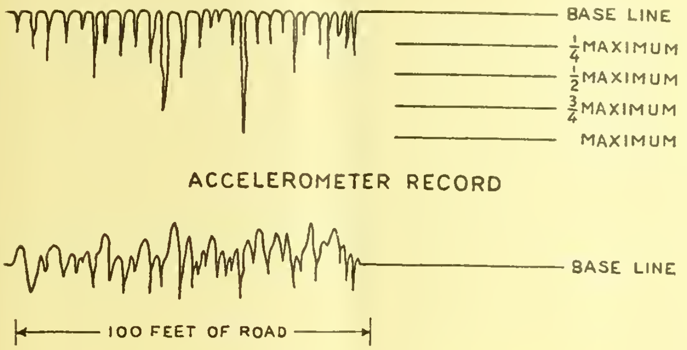
The truck spring under static conditions is deflected by the sprung load. At the instant of impact, the position of the truck spring above or below its average base line or static position is a measure of the sprung weight active at the instant of impact.

Test obstruction symbol	Unsprung weight				Sprung weight			Total vertical impact reaction	Total vertical impact reaction as a per cent of static load
	Accelerometer reading	Acceleration	Mass	Unsprung force	Spring reading	Variation from static	Sprung force		
	Inches	*Feet per second per second*	*Poundals*	*Pounds*	*Inches*	*Pounds*	*Pounds*	*Pounds*	*Per cent*
6s	0.08	162	22.5	4,370	0	0	1,625	5,995	255
6d	.25	506	22.5	12,110	.34	−1,605	20	12,130	516
7d	.18	364	22.5	8,915	.72	−3,400	−1,775	7,140	304
8s	.10	202	22.5	5,270	0	0	1,625	6,895	294
8d	.24	486	22.5	11,660	.40	−1,890	−265	11,395	485

The sum of the unsprung and sprung components is the total vertical road reaction. If this total impact reaction be divided by the total static reaction, it may be expressed as a percentage of the static load.

In the above example, the total static load per rear wheel is 2,349 pounds, of which 724 pounds is unsprung and 1,625 pounds is sprung. The accelerometer rate is 2,022 feet per second per second per inch of record, and 1 inch of truck spring record corresponds to a change of 4,720 pounds.

FIG. 7.—Analysis of typical record from artificial obstruction test



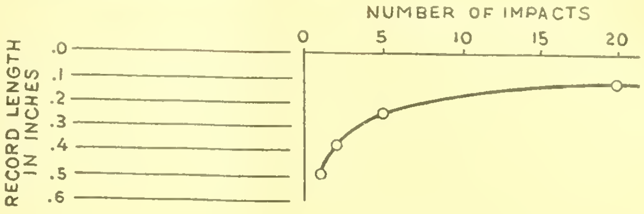
ACCELEROMETER RECORD

CORRESPONDING TRUCK SPRING RECORD

The force of the wheel in striking the road is developed (a) by the blow of the parts below and not supported by the truck springs and (b) by the weight of the parts above and supported by the truck springs.

(a) The blow struck by the unsprung weight is found by multiplying its mass by its acceleration. The mass is obtained by weighing, and the acceleration from the above record to which the acceleration of gravity is added.

(b) The sprung weight is obtained by weighing.



ACCELEROMETER ANALYSIS

Spring analysis

The average position of the truck spring is the same as when the truck is standing still, and the average pressure transmitted through the spring is equal to the weight of the body and load carried by each rear wheel.

For a wheel load of 9,000 pounds on a certain truck, the sprung weight was 7,800 pounds and the unsprung weight was 1,200 pounds, from which the unsprung mass was computed to be 37.3. The accelerometer used had a rate of 1,040 feet per second per second per inch.

$$F=m(a+g)+P$$

Number of impacts	Accelerometer reading	Acceleration (*a*)	$(a+g)$	Mass (*m*)	$m(a+g)$	Spring pressure (*P*)	Total impact force (*F*)
	Inches	*Feet per second per second*	*Feet per second per sec.*	*Poundals*	*Pounds*	*Pounds*	*Pounds*
1	.50	520	552	37.3	20,600	7,800	28,400
5	.25	260	292	37.3	10,900	7,800	18,700
10	.18	187	219	37.3	8,200	7,800	16,000
20	.12	125	157	37.3	5,900	7,800	13,700

FIG. 8.—Analysis of typical record from highway test

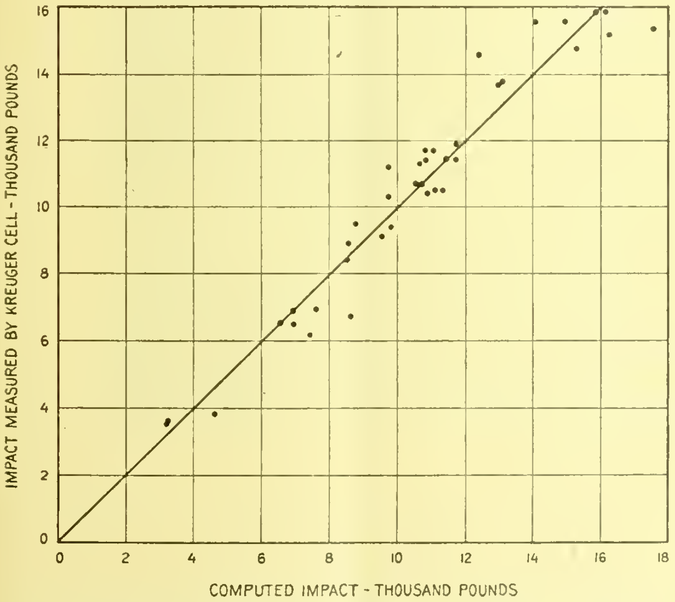


FIG. 9.—Diagram showing relation of impact reactions as computed from measured accelerations and as determined directly by means of the Kreuger cell

Calibration factors for various accelerometer spring-weight combinations have been determined for use with the full range of tire equipments included in the tests. These factors have been established analytically, indirectly, and directly. Formulæ, substantiated by experimental data, have been developed for the determination and checking of these calibration factors. The instruments have been calibrated by relating the instrumental records of impressed accelerations with determinations of the same accelerations from analyses of space-time records. That method of space-time analysis which assumes a parabolic shape of the original space-time curve over an appreciable period was selected as representing the average constant maximum acceleration. Should the instantaneous maximum be desired, it can be obtained from the calibration data already on hand and will vary from 100 to about 115 per cent of the parabolic maximum, according to conditions within the test range.

The accelerations were measured under two general conditions described by the terms "shock" and "drop." The condition of shock is defined as the reaction between the truck wheel and the pavement when the wheel is caused to pass over an elevation in or obstruction on the pavement. The condition of drop is defined as the reaction occurring when the wheel, falling under the combined influence of gravity and the truck spring, is suddenly stopped by the pavement.

On the above basis, for the test conditions to date, the errors in the measurement of acceleration will generally be within 10 per cent for all conditions of drop and for conditions of shock up to 10 miles per hour. For conditions of shock between 10 and 15 miles per hour, the error may be 15 per cent, and between 15 and 20 miles per hour for the condition of shock it may be 20 per cent. These values are considered conservative and are based on calibrations made by a special accelerometer subcommittee, which covered adequate ranges of speed and obstruction with pneumatic and new solid tires. The complete procedure and results of the accelerometer calibration are to be made the subject of a separate paper. Reference is made to an article in the December, 1924, issue of Public Roads by L. W. Teller, entitled "Accurate Accelerometers Developed by the Bureau of Public Roads."

After the accelerometer had been developed by the bureau and before it was used in the tests, it was checked by measuring the road reaction directly and comparing it with the computed reaction. The direct measurement was made by means of a Kreuger cell [1] placed in a pit so as to be squarely under the truck wheel at the instant of impact. The results of this check test are shown in Figure 9.

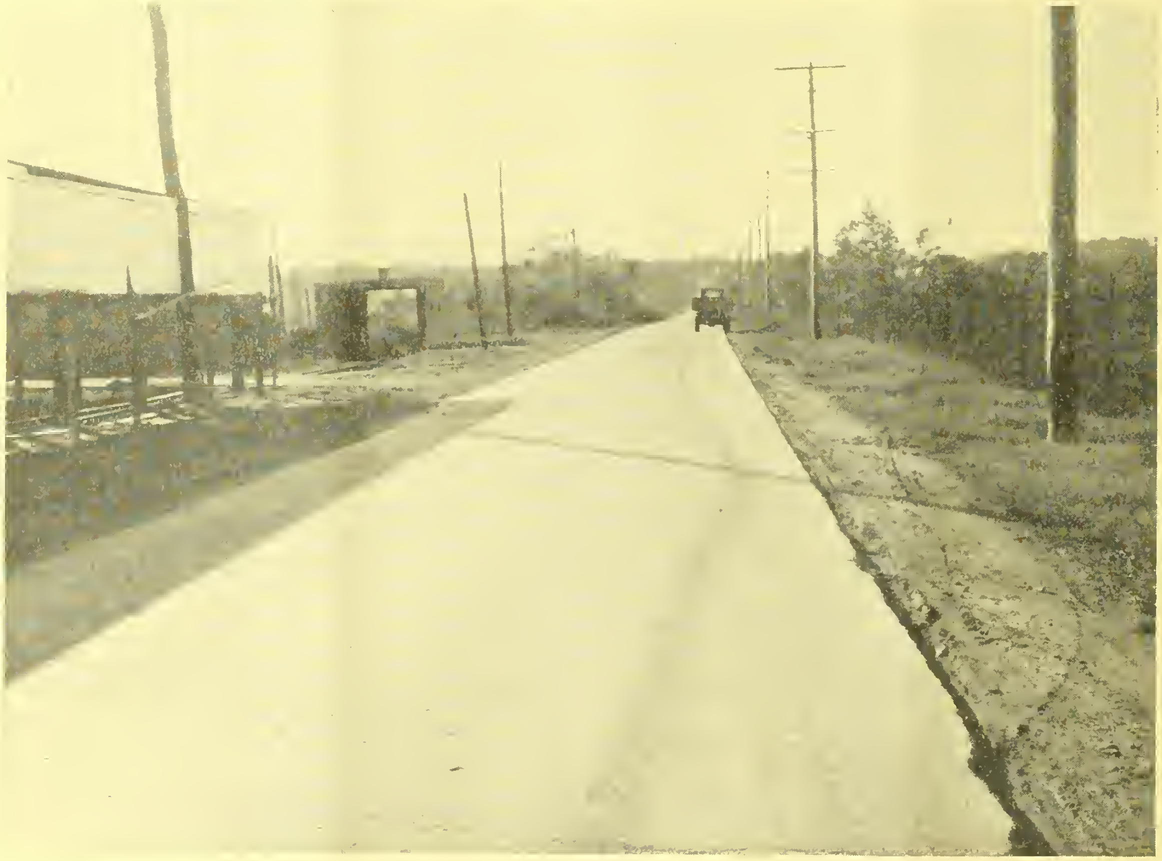


FIG. 10.—The smooth concrete test section

TIRES, SPRINGS, TRUCK SIZES, AND SPEEDS

The tires used were furnished by seven American manufacturers. The types were pneumatic, cushion, heavy-duty cushion, and high-profile, nonskid and regular solid tires. Tests have been made with new and worn-out tires, and with new tires cut to various heights. The pneumatic tires were tested at standard and three-fourths standard air pressures. The sizes varied from 32 by 3½ to 40 by 14, and the rated carrying capacities varied from 1,300 to 11,200 pounds per tire. They were used in single, dual, and tandem mountings, and as overloaded and oversized equipment.

The trucks used represent four American makes, the capacities of which are 1, 2, 3, and 5 tons. They were loaded up to 150 per cent of the carrying capacity of the standard types and sizes of tires with which they were equipped, using for the purpose 100-pound weights which were placed within the truck body in a definite manner and according to accepted practice in load distribution. The wheel loads were measured with portable road scales, which were frequently calibrated. In addition, a number of loads were sent to accurate platform scales for checking.

The trucks were driven at speeds varying by increments of 3 miles per hour from a minimum of 3 miles per hour up to the maximum speed obtainable under the load and test conditions. Speeds were determined by timing measured runs with a stop watch, and were checked for variation by the truck speedometer. Any run which showed a variation of more than one-half mile per hour from the average speed was discarded and the test was rerun.

Before use in the tests the rear truck springs were removed, cleaned, greased, calibrated, provided with jackets, and replaced. For reference, weight distributions of the empty trucks, truck measurements, and specifications were recorded.

ROAD SURFACES AND ARTIFICIAL OBSTRUCTIONS

In order to include actual road conditions, two 500-foot lengths of near-by highways were selected as test-road sections to represent extremes likely to be encountered by commercial traffic. One of these is typical of smooth concrete construction, and the other is an old stone-block pavement. A 16-wheel profilometer showed an average cumulative vertical variation of 2.5 inches per 100 feet on the smooth concrete section and of 18.2 inches per 100 feet on the rough stone-block section. Both sections are straight and substantially level, and have only slight crowns. In general, the tests on these two sections were confined to tire capacity and 1½ tire capacity loads at a representative speed of 12 miles per hour. The results of these tests are expressed in terms of the number and magnitude of the impacts (i. e., vertical road reactions) produced as determined from the accelerometer records. A photograph of the stone-block pavement appears on the cover of this issue of Public Roads, and the concrete road is shown in Figure 10.

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[1] For a description of the Kreuger cell and its use in measuring impact reactions see Public Roads, vol. 5, No. 10, Dec., 1924.

To provide for duplication of test conditions, a very smooth, straight, and substantially level concrete road was specially built, and on it various steel obstructions were set in grout and secured by bolts. These obstructions included 30-inch inclined planes with rises of $\frac{13}{16}$, 1½, and $1\frac{15}{16}$ inches, respectively; rectangular blocks 3 inches wide and with heights of $\frac{9}{16}$, ⅞, and 1⅛ inches, respectively, and rounded (segmental) blocks having 3-inch chords or bases and heights of ¾ and 1½ inches, respectively. These obstructions were about 4 feet wide and were placed at about 30-foot intervals along a guide line painted on the road. Figure 11 shows typical obstructions.[2]

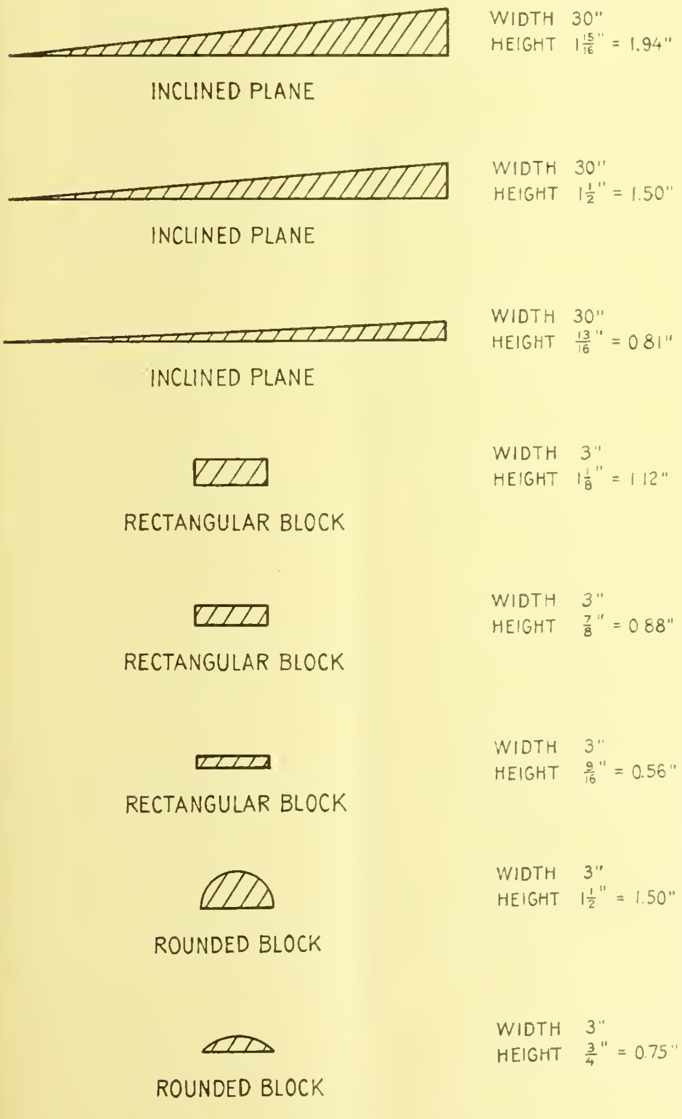
WIDTH 30"
HEIGHT $1\frac{15}{16}$" = 1.94"
INCLINED PLANE
WIDTH 30"
HEIGHT $1\frac{1}{2}$" = 1.50"
INCLINED PLANE
WIDTH 30"
HEIGHT $\frac{13}{16}$" = 0.81"
INCLINED PLANE
WIDTH 3"
HEIGHT $1\frac{1}{8}$" = 1.12"
RECTANGULAR BLOCK
WIDTH 3"
HEIGHT $\frac{7}{8}$" = 0.88"
RECTANGULAR BLOCK
WIDTH 3"
HEIGHT $\frac{9}{16}$" = 0.56"
RECTANGULAR BLOCK
WIDTH 3"
HEIGHT $1\frac{1}{2}$" = 1.50"
ROUNDED BLOCK
WIDTH 3"
HEIGHT $\frac{3}{4}$" = 0.75"
ROUNDED BLOCK

Fig. 11.—Typical obstructions used in the tests

An almost unlimited number of comparisons may be made from the test data showing the influence of the several variables on vertical impact reaction. It is not possible in this report to present exhaustive data with reference to all conditions studied; but general comparisons are made in the curves presented in Figures 12 to 31, and the original data in plotted form and the standardized computations are accessible to those who have a proper interest in making detailed studies or comparisons. In presenting this report the data are submitted as found, and the curves faithfully follow their plotted points. The ordinates of all the comparative curves represent the impact force expressed in terms of percentage of static load; to convert them into pounds it is necessary to multiply them by the corresponding static wheel loads in pounds according to Table 1. The overloaded and oversized tire tests were made at loads corresponding to 1½ the capacity of the rated sizes of tires.

Table 1.—*Wheel loads and tire sizes*

Tire type	Truck capacity	Wheel load capacity of rated size of tire		Tire size			
				Rated size		Oversize	
		Single	Dual	Single	Dual	Single	Dual
		Pounds	*Pounds*	*Inches*	*Inches*	*Inches*	*Inches*
Pneumatic	2 tons	4,000	4,400	40 by 8	36 by 6		38 by 7
	3 tons		6,000		38 by 7		40 by 8
	5 tons		8,000		40 by 8		
Cushion	2 tons		3,400		36 by 5		36 by 7
	3 tons		7,000		36 by 7		36 by 8
	5 tons						
Solid	2 tons		4,000		36 by 4		36 by 6
	3 tons		6,000		36 by 5		36 by 7
	5 tons	8,400	8,400	40 by 12	40 by 6	40 by 14	40 by 8

EFFECT OF ROAD ROUGHNESS

Figures 12 and 13 show typical results obtained on the highway test sections. They show that pneumatic tires may make a rough road appear reasonably smooth and that solid tires in poor condition may make a smooth road appear unreasonably rough. Referring to the two figures we find, for instance, that with tires loaded to capacity and trucks operated at 12 miles an hour, the impact magnitude which occurs 50 times in 500 feet, when pneumatic tires are used, is 5 per cent greater than the static load on the smooth road and 40 per cent greater than the static load on the rough road. With worn-out solid tires on the two classes of road the impact occurring 50 times in 500 feet was 45 per cent greater than the static load on the smooth road and 270 per cent greater than the static load on the rough road. These curves also show that the forces developed on a road which is built and maintained smooth are, for given equipment, appreciably lower than the corresponding forces developed on a very rough road.

The impact road reaction of a motor truck depends in general on the four major variables—wheel load, truck speed, tire equipment, and road roughness or

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[2] No artificial ruts or pot holes have been included to date in the truck tests. A few tests have been made with the impact testing machine which is described and illustrated on page 13 of the November, 1924, issue of Public Roads, by dropping the wheel upon a surface hollowed to a 40-inch radius. The results of the tests with this artificial rut did not seem to justify inclusion of such an obstruction in the truck tests.

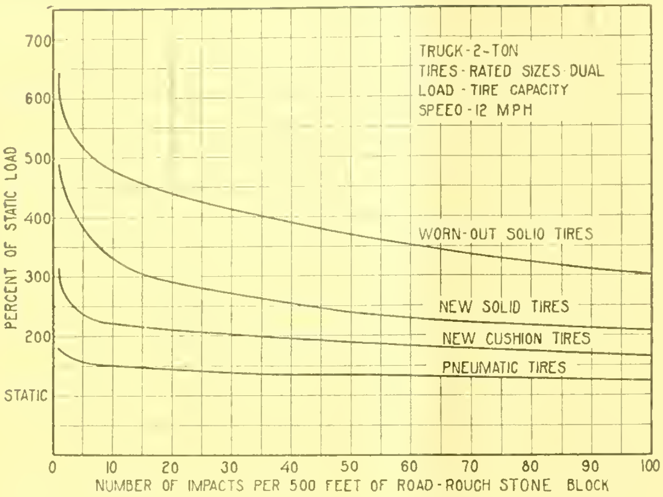

Fig. 12.—Influence of tires on vertical impact reactions on rough stone-block road

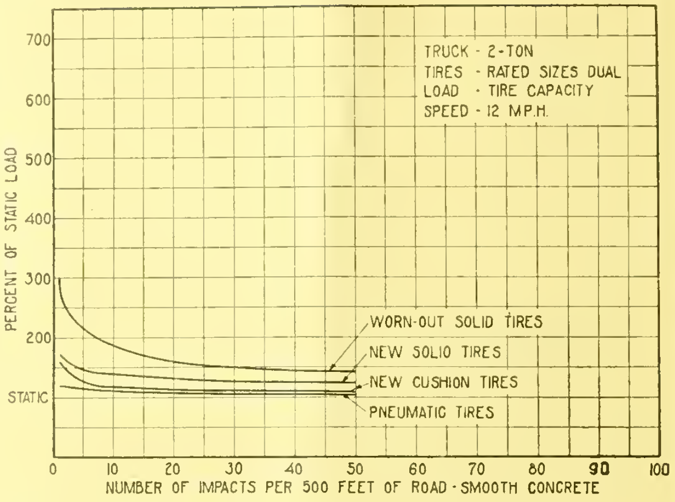

Fig. 13.—Influence of tires on vertical impact reactions on smooth concrete road

height of obstruction. The effects of other variables, such as the ratio of unsprung to sprung weights, truck spring flexibility, etc., are also felt to a greater or lesser degree, but manufacturing and operating conditions are generally such that the effect of these latter variables is not of major importance. The phenomenon of motor truck impact is separable into two distinct elements. There are two forces acting simultaneously and cumulatively on the pavement. One of these forces is the net truck spring pressure at the instant of impact, its magnitude varying with the spring deflection. The other is the force required to change the vertical velocity of the unsprung truck weight (that is, the parts below and not supported by the truck spring) as the wheel is vertically accelerated or decelerated by the pavement.

When the vehicle is standing still the road reaction is the static load on the wheel; but with the truck in motion this condition no longer obtains. What takes place when the wheel in motion strikes and passes over an obstruction on the road may be described in general terms as follows:

We will assume that the road is otherwise perfectly smooth and that the truck spring is at equilibrium in its normal position. As the wheel in its forward progress makes contact with the obstruction the tire is compressed and through its resistance to compression, varying according to the type of tire used, the unsprung weight of the truck is accelerated upward against the resistance of the truck spring reacting on the sprung weight of the truck body. The acceleration of the unsprung weight causes a reaction on the pavement the magnitude of which is measured by the product of the acceleration produced and the mass of the unsprung weight which is accelerated. Added to this reaction is the superimposed pressure of the truck spring which at this condition, called shock, is substantially that of the static sprung weight of the truck. The truck wheel then leaves, or tends to leave, the pavement and compresses the truck spring. The truck body, in response to this pressure, rises slowly upward and the entire rear end of the truck is momentarily completely in the air. Very soon, however, the upward inertia of the unsprung weight is overcome by the truck spring and the wheel descends upon the pavement to be decelerated through the elastic medium of the tire. Now, at this point the truck body is usually considerably above its normal position, the springs being correspondingly extended. The springs in this extended position cause a lesser superimposed pressure than the static sprung weight of the truck to be added to the second impact reaction caused by the deceleration of the truck wheel by the pavement. It may even be the case that the truck spring is so far extended that the resultant spring pressure is negative and actually lessens the effect of the deceleration of the unsprung weight. On the other hand it is possible, in the case of a long jump after striking the obstruction, for the truck spring to have fully extended and be in the act of closing again when the wheel strikes the road so that the sprung component may be even greater than the static spring pressure of the truck.

EFFECTS OF HEIGHT OF OBSTRUCTION, SPEED AND LOAD

In Figure 14 it will be noted that there is apparently a critical height obstruction, varying between 1 and 2 inches, according to the type of obstruction, at which the impact reaction approaches one of its maxima for the given tire equipment. The influences of the sprung to unsprung weight ratio, the truck load, and the truck spring characteristics are probably responsible for this condition.

Figures 14, 15, and 16 show typical results obtained from the artificial obstruction tests. The effects of three types of obstruction as influenced by height, and the influences of the speed of and load on the truck when passing over representative obstructions of these types are shown for typical tire conditions. It will be noted from Figure 14 that pneumatic tires did not yield reactions (under these test conditions) as great as two times the static load for any height of the inclined plane or for the 3-inch rectangular blocks, but that when worn-out solid tires were used, an impact reaction twice as great as the static load was caused by about 0.15 inch height for all three types of construction. A glance at this figure also shows that the impact reactions with worn-out solid tires may, under certain conditions, reach 11 times the static load; but that new solid tires show only about one-half of this increase over the static load.

In Figure 15 the influence of truck speed is shown. There is usually a critical speed between 12 and 15 miles per hour, at which, under the test conditions, the impact reaction will be at one of its maxima, particu-

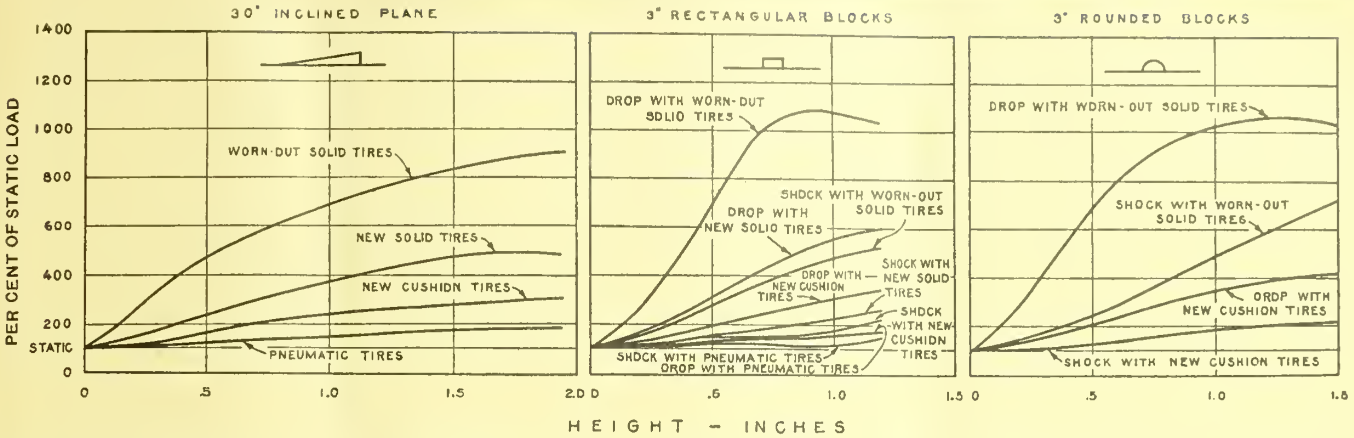

FIG. 14.—Effect of height and type of obstruction on vertical impact reaction. The tests were made with a 2-ton truck, equipped with rated-size dual tires, loaded to tire capacity, and operated at a speed of 12 miles per hour

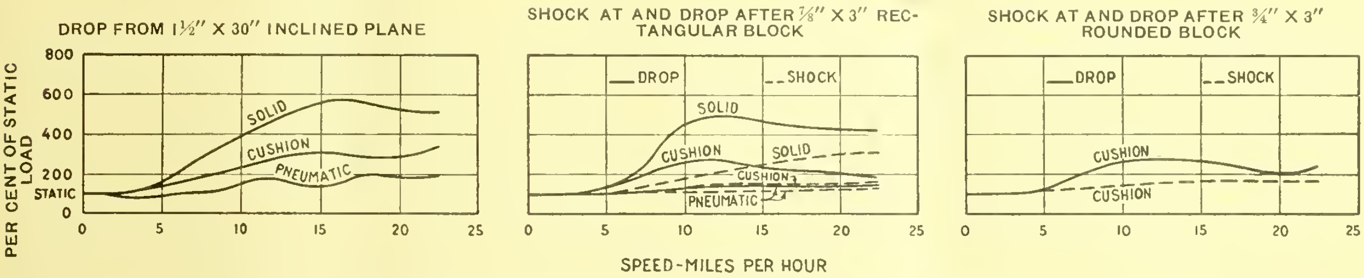

FIG. 15.—Effect of type of obstruction on vertical impact reaction as influenced by truck speed. These tests also were made with a 2-ton truck equipped with rated-size dual tires and loaded to tire capacity; and the truck was operated at various speeds

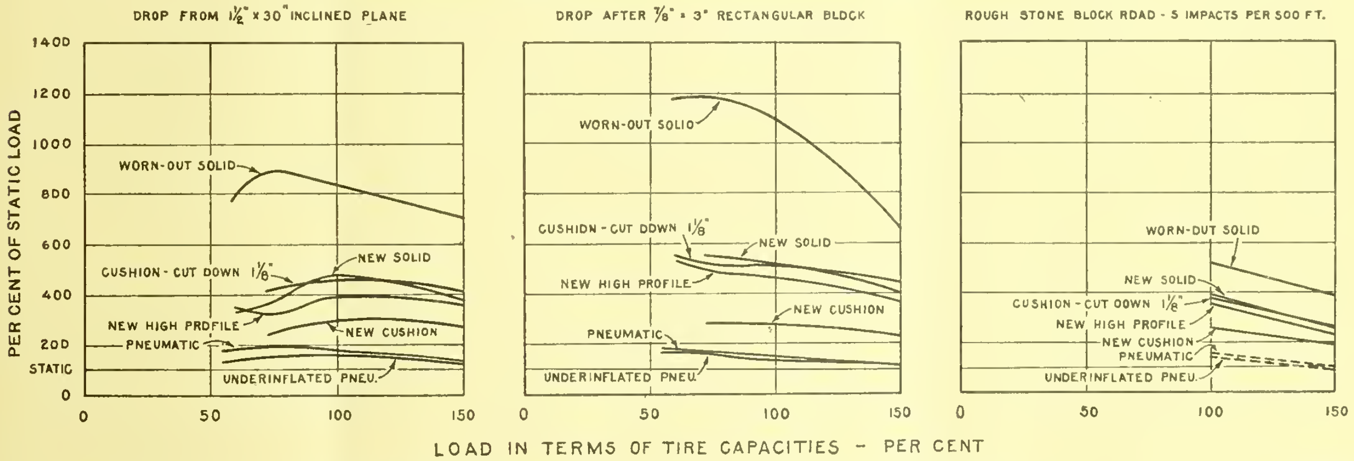

FIG. 16.—Effect of load in terms of tire capacity on vertical impact reaction with several types of tires and three conditions of impact. The curves represent tests made with a 2-ton truck equipped with rated-size dual tires, and operated at 12 miles per hour

larly for drop conditions. The reason for this is again found in the truck and load characteristics.

The most striking feature of the curves presented in Figure 16, which shows for various tires the effect of the load in relation to tire capacity, is the reduction in the impact with respect to the static load when the rated capacity of the tires is approached and exceeded. This phenomenon is probably due to the closer adherence of the truck wheel to the road under heavy load and also to the enforced cushioning of a greater part of the impact by the tires.

A clearer conception of the nature of the phenomena indicated by all three of these figures (figs. 14, 15, and 16) may be obtained by considering the behavior of two imaginary trucks; the first a truck of which practically the entire wheel load is derived from the unsprung weight and the body of which is but a skeleton supported by comparatively light springs. If, under such conditions, the tires were grossly overloaded, it will readily be seen that as long as there were any appreciable cushioning material present in the tires the excessive load directly on the tires would cause them, under all but the most extreme conditions of speed and road roughness, to remain in constant, intimate contact with the road. Comparatively slight obstructions would be embedded in the tire under the influence of the great load and there would be practically no difference between the cushioning qualities or springiness of various tires so long as they were able to carry the load at all. Now let us take another imaginary case in which the unsprung weight is a mere skeleton and the sprung weight, supported by adequate springs, is the principal load. An excessive wheel load, under these conditions, would not hold the tires so closely in contact with the road. The comparatively light wheel would be kicked upward as obstructions were encountered, the measure of the kick being determined by the cushioning properties of the tire. Having been raised into the air the wheel would, then, be returned to the pavement by the influence of the truck spring to be decelerated through the medium of whatever cushioning materials were in the tires. A truck under service conditions is a fortunate medium between the two imaginary extremes just described, but certain test conditions may be created, as shown by the figures, in which to a degree the effects described will be perceptible.

It is generally apparent from Figure 16 that as the static load increases the road impact reaction increases, but that the ratio of the impact reaction to the static load decreases.

EFFECTS OF TIRE EQUIPMENT

The next group of figures is concerned with tire equipments. Figures 17, 18, and 19 disclose the effects of the several variables on dual and single tire mountings, and Figures 20 and 21 show the relations between overloaded and oversized tire equipments. From an inspection of the three figures in the first group it is at once evident that the dual mounting of two smaller tires having approximately the same total carrying capacity as a larger tire mounted singly, causes a greater impact pressure on the road for a given condition. In the case of solid tires this effect of mounting may be such that the impact produced with dual tires will represent twice as great an increase over the static load as that produced with a single tire. The reason for this difference in impact forces caused by the two mountings is found in the cross sections of both the solid and pneumatic tires. The single tire, in each case, offers a thicker and narrower cushioning medium than the dual mounting. Consequently the acceleration or deceleration of the truck wheel is accomplished by the dual tire by less compression of the tire and therefore in a shorter time than a single tire. The acceleration or deceleration occurring in shorter time must necessarily be greater to produce the necessary change in vertical velocity for a given test condition. The acceleration of the unsprung weight being greater, the unsprung component of the road reaction is necessarily greater because it is the product of the acceleration times the mass of the unsprung weight. Given two otherwise equal cushioning media, that one which can effect the necessary change of velocity in the longer time interval will cause the lower impact force. These considerations lead to conclusions that:

1. Thickness and narrowness of tread rubber are desirable in reducing road impact reaction.
2. Increasing the thickness or profile height of rubber has a very marked effect in reducing road impact reaction in both single and dual mountings; and
3. In the tire equipments tested, all of which were standard at the time of the tests, dual mounting caused heavier impact forces than the corresponding single mounting of the same total load-carrying capacity.

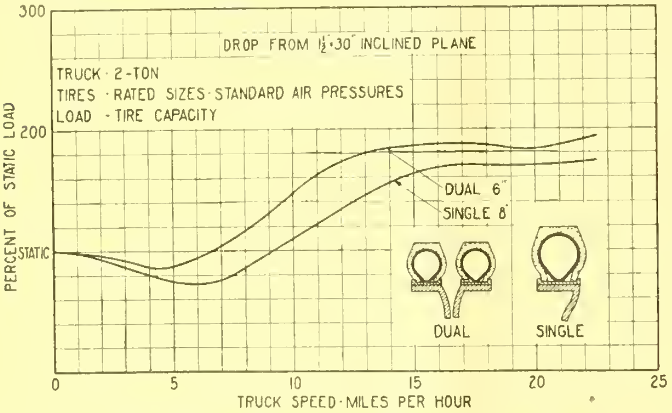

Fig. 17.—Single *vs.* dual tire mounting as influenced by truck speed

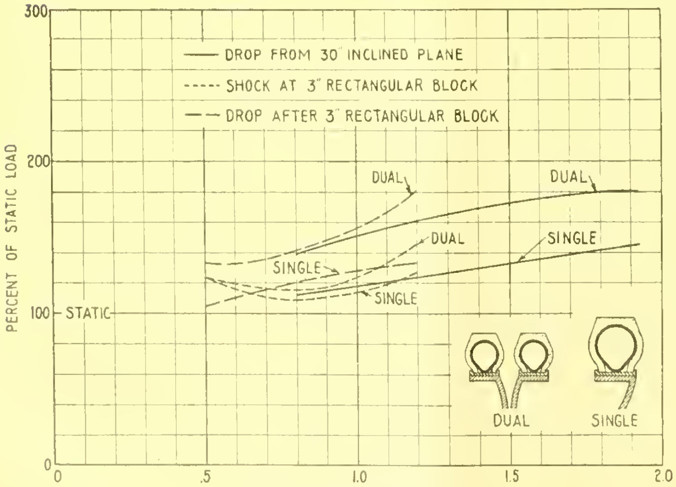

Fig. 18.—Single *vs.* dual tire mounting as influenced by height of obstruction

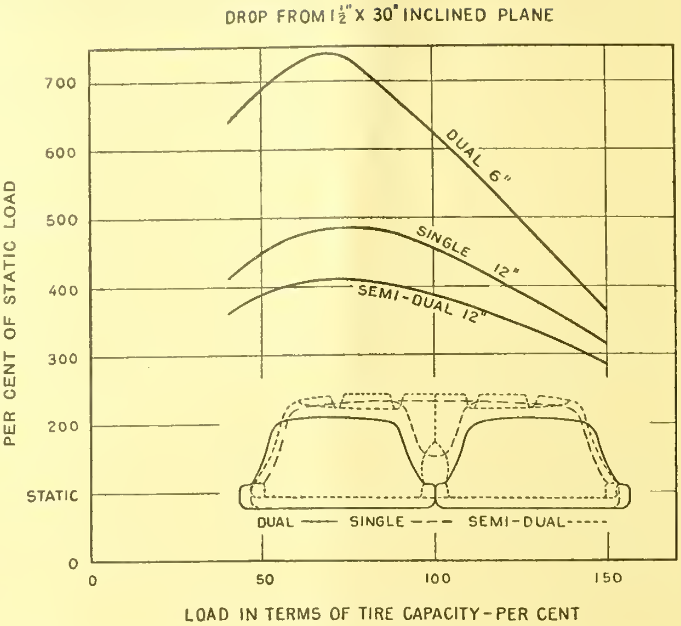

Fig. 19.—Single *vs.* dual tire mounting as influenced by wheel load. The tests were made with a 5-ton truck equipped with rated-sizes of tires and operated at a speed of 12 miles per hour

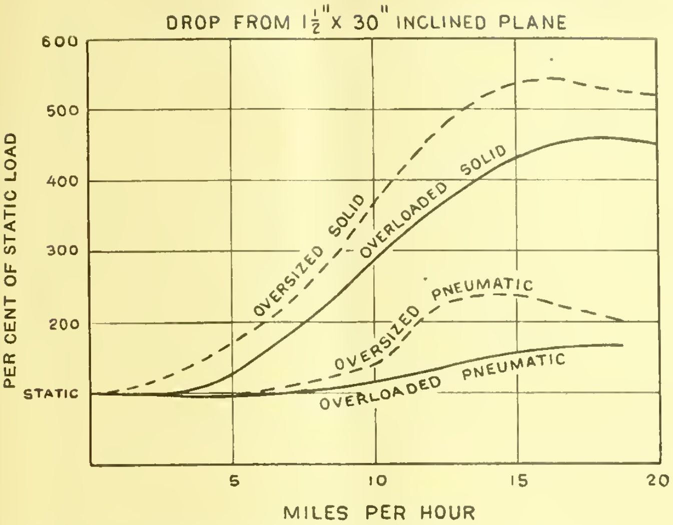

FIG. 20.—Effect of tire size on impact reactions resulting from drops from the 1½ by 30-inch inclined plane, using a 2-ton truck operated at various speeds. The overloaded tires carried a load equal to 150 per cent of their capacity, and the oversized tires carried the same load as the corresponding overloaded tires

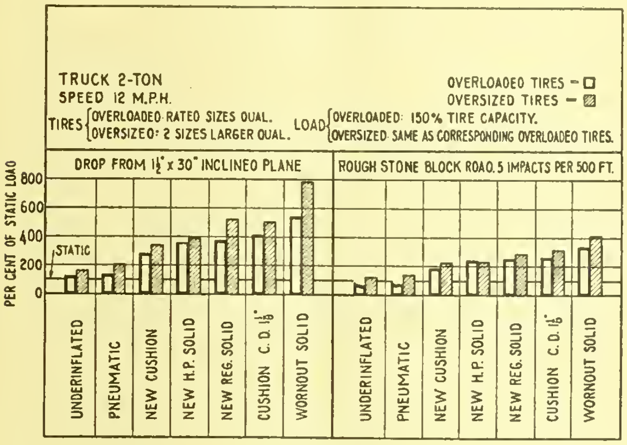

FIG. 21.—Relation between vertical impact reactions of overloaded and oversized tire equipments and the influence of tire type

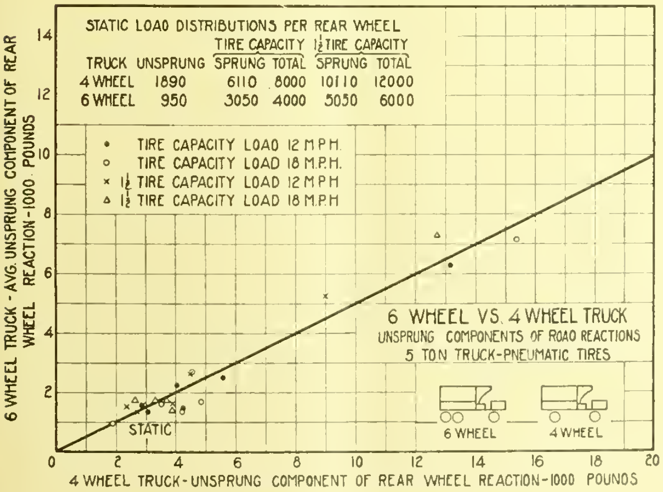

FIG. 22.—Impact relation of 6-wheel and 4-wheel trucks

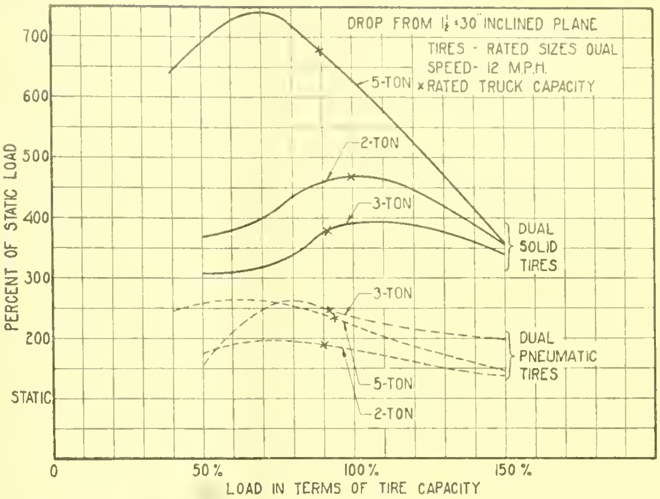

FIG. 23.—Vertical impact reaction as influenced by truck capacity and wheel load

Various relations between overloaded and oversized tires are brought out in Figures 20 and 21. It is at once apparent that, for truck wheels carrying the same load, those which are equipped with oversized tires cause heavier road impact reactions than those equipped with the rated and smaller sizes of tires. The reason for this is found in the dimensions of the two sizes of tires. If the cross sections of the two sizes of tires were superimposed it would be found that they differ markedly in width and comparatively slightly in height. The narrower tire would naturally be more easily deformed laterally, and because of this, it could effect the necessary change in velocity by being compressed a greater distance more slowly than the wider tire would be during a relatively short compression distance. As explained in the discussion of the dual and single mountings, the tire which decelerates the truck wheel from a given velocity comparatively slowly will cause a lower impact reaction on the road than the tire which necessitates a short, quick reaction. It is interesting to note, in Figure 21, that, the impact force is about 15 per cent greater than the static load for the rated size of underinflated pneumatic tires, and nearly 700 per cent greater for the worn-out solid tires, which were oversized equipment for the same truck and at approximately the same load. The relative actions of these overloaded and oversized tires supply further support for the conclusions with respect to the effect of tires stated above.

SIX AND FOUR WHEEL TRUCKS COMPARED

In order to determine the comparative effects of 6 and 4 wheel vehicles, tests were made with two 5-ton trucks which differed only in the rear-wheel arrangement. One was a standard 4-wheel truck equipped at the rear with dual pneumatic tires. The other was the same make and size of truck but was equipped with two rear axles which carried singly-mounted tires of the same cross section as those carried by the 4-wheel truck. Both rear axles were driven by the motor through tandem differentials and the distance

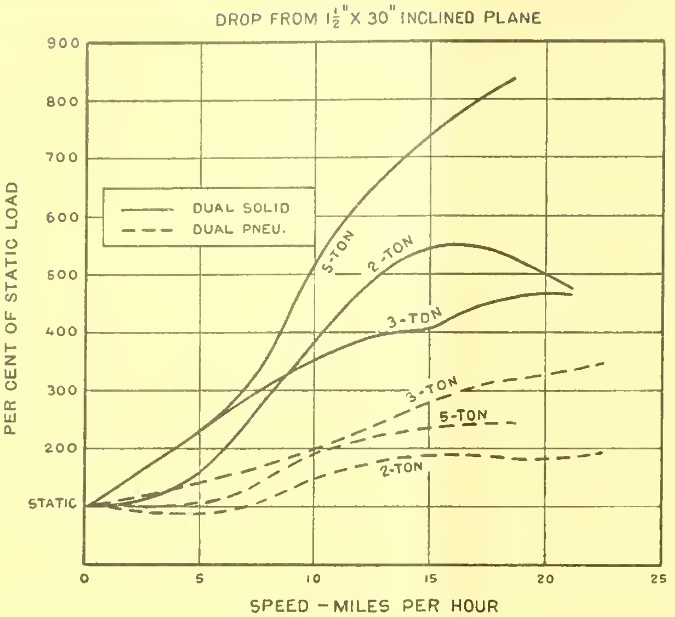

FIG. 24.—Effect of truck capacity on vertical impact reaction as influenced by truck speed. The trucks were equipped with tires of rated-size loaded to capacity

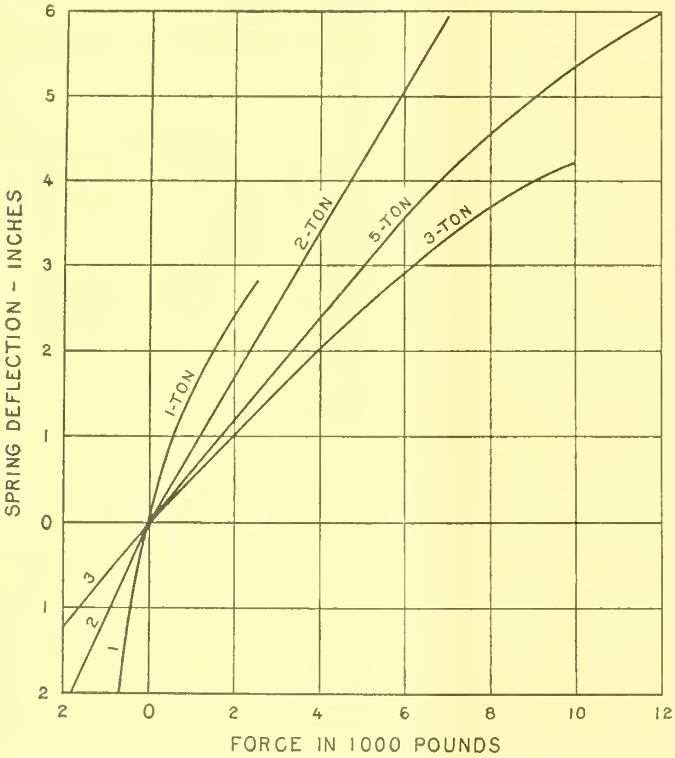

FIG. 25.—Truck-spring calibration curves, indicating the characteristics of the springs of the several test trucks

between axle centers was slightly over 4 feet. The usual truck spring was inverted, one spring being between the two axles on each side. The 6-wheel truck carried on its two rear axles a total load equal to that carried by the 4-wheel truck on its one rear axle.

In Figure 22, the unsprung components of the impact reactions of the 4-wheel truck are shown to be twice those of the equivalent 6-wheel truck, carrying the same load and operating under the same test conditions. This impact relation is particularly interesting in view of the results of other tests by the

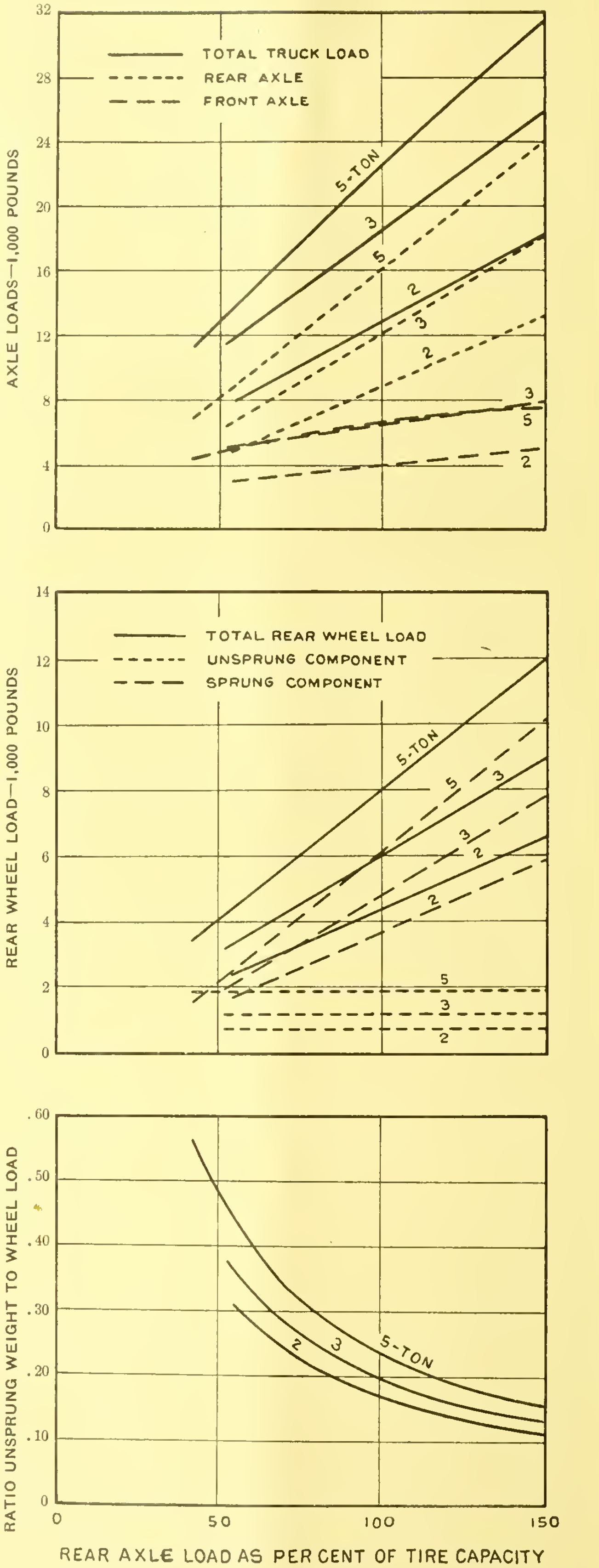

FIG. 26.—Diagrams showing distribution of the load of the various trucks equipped with rated sizes of dual pneumatic tires as used in the tests

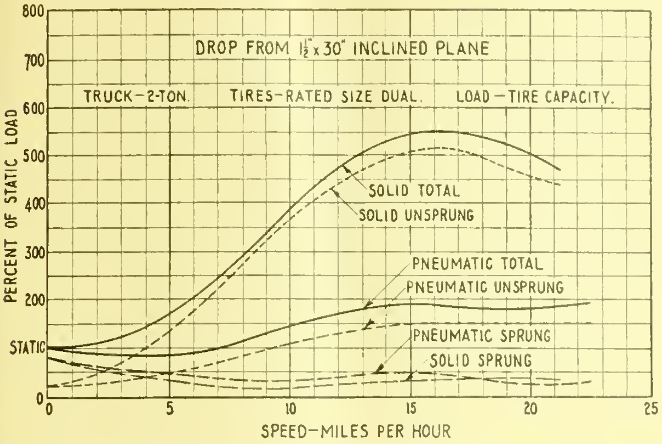

FIG. 27.—Components of the vertical impact reaction as influenced by truck speed

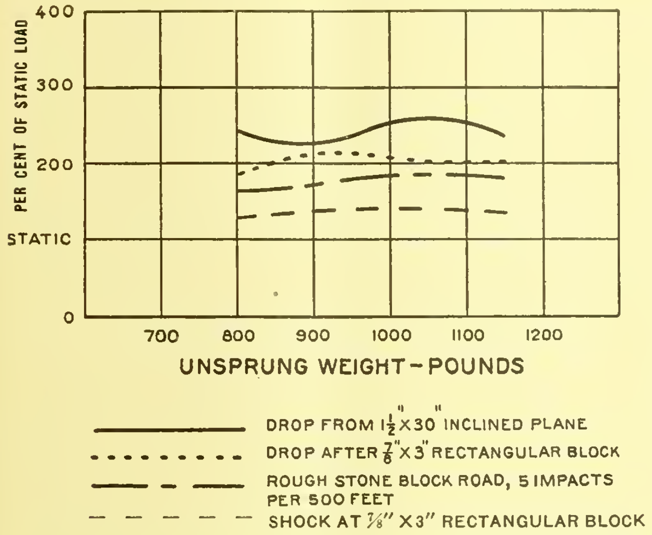

FIG. 28.—Curves showing the effect of varying the unsprung weight of a 2-ton truck equipped with new 5-inch dual cushion tires and operated with a rear-wheel load of 5,100 pounds at a speed of 12 miles per hour

bureau reported in the article entitled "The Six-Wheel Truck and the Pavement" which appeared in the October, 1925, issue of PUBLIC ROADS. These tests showed that the stresses in a concrete pavement caused by a 4-wheel truck are approximately twice as great as those caused by a 6-wheel truck carrying the same load.

Figures 23, 24, 25, and 26 are concerned with truck conditions. Figure 23 shows the effect of truck capacity and the influence of wheel load for the representative impact test conditions of the drop from a 1½ by 30 inch inclined plane at 12 miles per hour, with dual solid and dual pneumatic tire equipments. The rated truck capacities in terms of tire load are indicated and it will be seen that, with one exception, the trucks were overtired when equipped with sizes ordinarily specified as standard equipment, the trucks being fully loaded when the tires were carrying about 90 per cent of their rated capacities. It will also be noted that excessive overloading has a tendency to eliminate effects of individual truck characteristics because under the extreme sprung loads the truck springs are not permitted to function in a normal manner. As indicated in Figure 23, the 2, 3, and 5-ton trucks are about to seek a common curve to represent their behavior as the load increases for a given type of tire equipment. This does not mean that the reactions in pounds for each of these trucks are identical, but that the relative effects which are expressed as percentages of the static load are approximately the same. As previously discussed, these curves also show that excessive loads have a great leveling effect on cushioning qualities whether of the truck spring or tire equipment, and if the tires (with cushioning abilities between those of the new solid and the pneumatic types) were loaded to two and one-half or three times their rated carrying capacities it would appear that the increase over static load would become negligible for the given speed and obstruction conditions. The influences of the individual trucks and their spring characteristics are more pronounced in Figure 24, which shows the influence of the speed of the trucks. As tending to explain some of the apparently erratic indications of Figures 23 and 24 it should be stated that the truck which is designated as of 3 tons capacity was in reality a 5-ton chassis equipped with a 3-ton body. The characteristics of the springs are shown by the calibration curves in Figure 25 and the load distributions of the several trucks are given in Figure 26.

The vertical impact reaction separated into its components, caused by the sprung and unsprung parts is shown for a typical test condition in Figure 27. It has been found that the most severe impacts are obtained either at the shock on striking an obstruction or at the first rebound or drop after the obstruction has been passed. It is also true generally that the unsprung component of the road reaction is the major quantity, and, so long as the unsprung truck weight is decelerated (or accelerated) at all, it is likely to be the deciding factor in the total road reaction regardless of the compression of the truck spring.

The influence of the truck spring is felt, however, as shown in Figure 27, and under certain conditions, particularly at low speeds and with pneumatic tire equipment, the lessening of the sprung component due to the opening of the truck spring may be sufficient to cause the total vertical road reaction to be less than if the truck had been standing on the road.

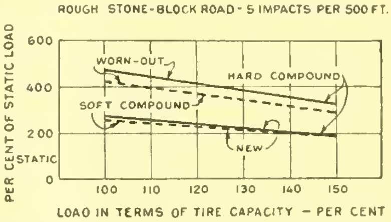

FIG. 29.—Effect of tire hardness on road reaction using new and worn-out tires

RESULTS OF AUXILIARY TESTS

The next group of figures shows results of a few auxiliary tests which have been made. Figure 28 shows the effect of varying the unsprung weight of a 2-ton truck equipped with 5-inch dual cushion tires and operated with a rear-wheel load of 5,100 pounds at a speed of 12 miles per hour. The unsprung weight was varied by lead weights affixed symmetrically to the rear axle by means of U bolts. Although this study is not exhaustive, it indicates that minor variations of the unsprung weight ratio do not seriously influence the road reactions. Figure 29 shows the effect of tire hardness or compound on the road reaction. A 3-ton truck was operated at 12 miles per hour with its rated-size, dual, solid-tire equipment. The tires were practically the same in cross section but varied in the hardness of rubber compound. They represent two standard makes and the tests indicate that the composition of tires in current use is not an important factor in impact reactions. Figure 30 shows the effect of internal tire temperature up to 190° F. for a typical test condition. A slight drop in the impact reaction is noted as the tire becomes warm. It was found to be impracticable to attain higher internal tire temperatures in the truck tests, and it is therefore not known what might occur under certain service conditions, wherein tires may reach their melting temperatures.

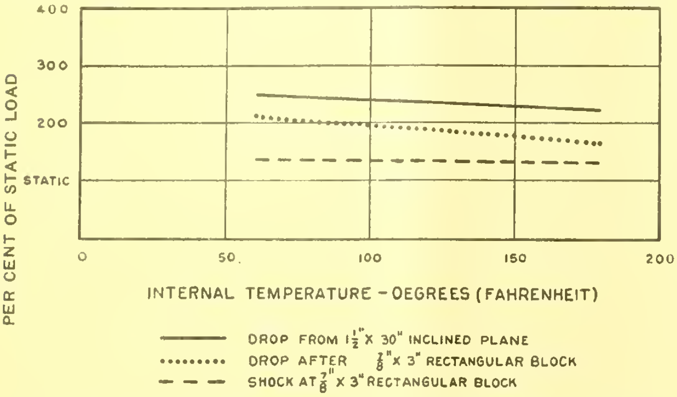

FIG. 30.—Effect of internal tire temperature on vertical impact reaction. The new tires used were 36 by 5 dual cushions. The truck was of 2 tons capacity and the load per rear wheel was 5,100 pounds. The unsprung weight per rear wheel was 1,116 pounds; and the speed was 12 miles per hour

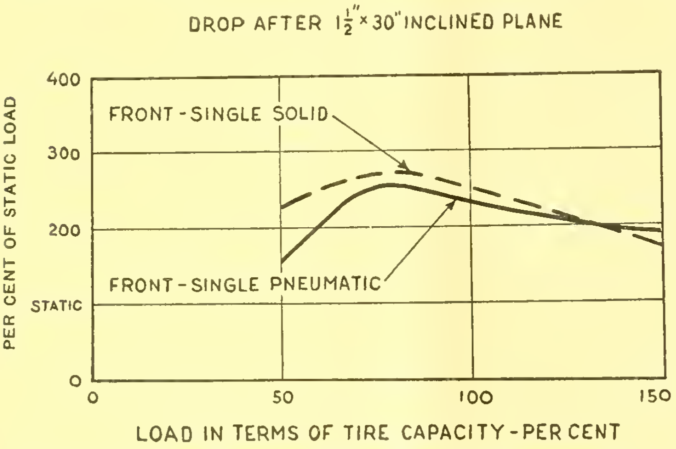

FIG. 31.—Effect on impact reaction of varying front-wheel tire equipment as influenced by the rear tire load. This test was made with a 3-ton truck, having rear wheels equipped with rated-size dual pneumatic tires, and operated at 12 miles per hour over the 1½ by 30-inch inclined plane

The influence of single pneumatic and single solid-tire equipments on the front wheel of a 3-ton truck the rear wheels of which were equipped with dual pneumatic tires is shown in Figure 31. The truck was operated at a speed of 12 miles per hour over the 1½ by 30 inch inclined plane. This study, though not exhaustive indicates that the front wheel equipment does not greatly influence the rear wheel reaction.

During the tests a certain type of new tire, which had a nonskid tread design now obsolete, was used on a truck as dual equipment. The tires were placed on the wheel with the tread designs opposite and then with the tread designs staggered. The markings on these tires were unusually deep and by varying the mountings as above an exceptional variation in cross-sectional rubber was obtained. This change in mounting was of itself sufficient to reduce the impact reaction from 3,600 to 1,200 pounds at each repetition of tread design (or about 8 inches) along a smooth concrete road.

From the tests made on these tires it is concluded that any appreciable variation of cross-sectional rubber or breaks in its continuity will cause heavy repeated impacts to be delivered to the road; and it is specifically recommended that dual tires should always be mounted with the tread designs staggered.

New French Magazine Appears

Revue Générale des Routes, to be Published Under the Auspices of the Ministry of Public Works

Announcement has been received of the publication in France of a new monthly magazine, the Revue Générale des Routes et de la Circulation Routière, under the auspices of the road office of the Ministry of Public Works.

Introduced by Senator Albert Mahieu, president of the permanent international association of road congresses, the new magazine has the backing of all groups interested in the solution of French road problems, including members of Parliament, the national administration, contractors, highway engineers, producers of road materials, and representatives of the motor vehicle industry.

The aim of the publication, as announced by M. Pierre Guieu, director general, is to contribute in all possible ways to the reconstruction of the French road system and to the improvement of methods of highway administration, financing, and construction.

In addition to the introduction by Senator Mahieu the first number contains interesting articles on grade crossings, slippery pavements, city street construction, and a technical index, including a bibliography, extracts from reviews, and a list of patents by recognized French authorities on the various subjects.

In subsequent numbers the discussion of slippery pavements and grade crossings will be continued; and among other articles promised is one on American roads by A. Antoine. Others will deal with the protection of dangerous crossings, studies of road materials, and the construction of forest roads. The forthcoming issues will also contain full information with regard to the Fifth International Road Congress at Milan, including programs and itineraries.

The offices of the new publication, of which M. Pierre Guieu is director general, are at 9 Rue Coëtlogon, Paris (VI). The foreign subscription rate is announced as from $5 to $6.

MAXIMUM STREAM FLOW

A FORMULA FOR GENERAL USE

By **C. E. GRUNSKY**, Consulting Engineer and Past-President American Society of Civil Engineer

THE FOLLOWING comments on maximum stream flow are prompted by the article published in PUBLIC ROADS, February, 1926, by Capt. C. S. Jarvis. This article is entitled "A General Formula for Waterways" and its author advocates the use of a formula for maximum stream flow based on area and a variable coefficient. He has selected the Myers formula in which maximum discharge is assumed to be proportional to the square root of the watershed area as best adapted to express the increase of discharge as area increases. He quotes in comparison formulas for the maximum rate of discharge as devised by Fanning, Talbot, and McMath.

All of these formulas have inherent defects and although applicable under certain conditions and in localities where data are available for determining the coefficients they obviously lack the elements which would make them generally applicable. The reasons why this must be so will be briefly set forth.

The intensity of the rainfall and its distribution throughout any given watershed is one of the main factors which determines the maximum rate of run-off at the outfall point of the watershed. But maximum rain intensity has a wide range throughout the world. It is largest on small areas. It decreases, in other words, as area increases. The run-off producing storm may, moreover, have a materially different effect if the storm travels downstream from that which it would produce if it traveled upstream.

The outline of the watershed, whether compact or elongated, broad at the base or narrow at the base, affects the rapidity of water concentration and therefore the peak run-off rate.

The gradient of the main lines of flow along which the water moves to arrive at the outfall determines the time which it takes the water from the various subdivisions of the watershed to get to the outfall point, and, therefore, the topographic and geographic features of the watershed must be regarded as having an effect upon the maximum rate of run-off.

Finally there is the effect of the size of the watershed on this rate. The maximum rate of rainfall, considered as an average over large areas, grows smaller as the areas in question grow larger; and in consequence the rate of run-off per unit of surface must likewise be smallest for the largest area.

It follows that every formula of the type of those which are noted by Captain Jarvis in his contribution are seriously at fault because they do not contain a rain-intensity factor. They are serviceable at the best only when the areas to which they are applied are similar in general topographic characteristics, in outline, in geologic surface structure, and in meteorological conditions. When this is the case, and then only, can such formulas be used, with some confidence, to approximate the maximum stream flow from watersheds of various areas, based always, however, on the known maximum stream flow from at least one area of similar characteristics which will serve to establish the value of the variable coefficient which appears in the formula.

GENERAL FORMULAS MUST INVOLVE RAINFALL INTENSITY

Any formula for general application must take into account the intensity of the rainfall which produces maximum stream discharge. This intensity when applied to an area of some extent, in contradistinction to a single point, decreases as the area increases. Furthermore the larger the areas of similar shape, the longer will it take the run-off to reach the outfall point. And, moreover, less time will be required for run-off waters to reach an outfall point from a fairly circular compact area than from an area which is much elongated.

It follows that the time which it takes the run-off to reach the outfall point from the various subdivisions of a watershed is an important element to be considered in devising a formula for the maximum stream discharge. Whenever it is possible to approximate the time (*t*) required for the maximum concentration of run-off water at the outfall point, it is possible also to compute from the known weather or meteorological conditions the volume of the precipitation in this concentration period, and this volume of water can then be made the starting point in the estimate of storm-water discharge. The further modifying factor depending on the absorptive properties of the surface of any area in question must also be taken into account and, because these surface conditions are variable, will necessarily appear in any formula as a variable. The effect of the retention of water in depressions or basins, such as lakes, swamps, and the like, when these are relatively small, will generally be sufficiently taken care of in the determination of the time to be allowed for run-off concentration. When a relatively large area of a watershed is lake or swamp surface, special consideration must be given to the effect of storage on the run-off and on the maximum stream flow, and ordinary formulas will no longer apply.

A rather complete analysis of this subject has been attempted by the writer, the results of which were contributed to the American Society of Civil Engineers.[1] It will not be necessary to review this analysis which showed the intimate relation of maximum stream discharge to maximum precipitation intensity. It will suffice to say that it led to the following suggestions:

1. The maximum rainfall intensity can, for all practical purposes, be expressed by the formula

$$I=\frac{C}{\sqrt{t}} \qquad (1)$$

where I represents the maximum rain in inches per hour during the time t (in minutes), and C is a constant for any locality to be determined from the known maximum rate of rain in one hour, whenever small areas are involved, and from the maximum rain in some longer period, as, for example, 24 hours when large areas are involved. Thus, for example, if the maximum rainfall R, in one hour (60 minutes) is 1 inch then

$$R=\frac{C}{\sqrt{60}} \text{ and } C=\sqrt{60}, \text{ or } C=7.75.$$

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[1] The Sewer System of San Francisco, and a Solution of the Storm-water Flow Problem. Trans. Amer. Soc. of Civ. Engrs., Vol. LXV, p. 294; and Rainfall and Run-off Studies. Trans. Amer. Soc. of Civ. Engrs., Vol. LXXXV (1922), p. 66.

This is a value which will apply to some Pacific coast conditions. Or again if it be known that the maximum precipitation in 24 hours over some large area in question is 4 inches then

$$I_{24}=\frac{4}{24}=0.167 \text{ and } 0.167=\frac{C}{\sqrt{1,440}}, \quad \text{making } C=6.3$$

In the Middle West and on the Atlantic coast the values of the constant in the rainfall intensity formula will generally be three to five times as great for small areas and two to three times as great for large areas as on the Pacific slope.

2. The formula for maximum storm-water flow should take into account this maximum possible rain intensity and the topographic and soil characteristics of the watershed. Such a formula should, therefore, include as one of its elements rainfall intensity as deduced for the region in question from meteorological conditions, and the time of run-off concentration, and, also, a variable factor based on the soil characteristics of the watershed.

A GENERAL FORMULA RECOMMENDED

These considerations have led the writer to adopt and recommend a formula for maximum stream flow which is as simple as could well be desired and which will give dependable results whenever adequate data relating to rainfall and the surface characteristics of the watershed are known. The Jarvis modification of the Myers formula reduces it to a mere skeleton, making the maximum discharge proportional to the square root of the area, but retaining therein a factor *p* or *P* which the engineer must evaluate on some information relating to the same factors which enter into the type of formulas intended for more general application. The principal purpose of the Jarvis formula, and this only within certain limits, seems to be to approximate discharge from a watershed when the discharge from some other watershed of similar surface character and in the same general region is already known.

Other formulas such as the McMath formula and the Burkli-Ziegler formula [2] take rainfall intensity into account. They are in a different class from the Myers formula as modified by Captain Jarvis and such other formulas as those of Kuichling, Fuller, and Metcalf and Eddy in which the precipitation factor does not appear. But even the McMath and Burkli-Ziegler formulas have their limitations which it is not proposed to discuss at this time.

The formula which the writer suggests as one for general use is as follows:

$$D_m=413\,aMI \tag{2}$$

In this formula D_m=maximum rate of discharge from the watershed in second-feet.

M=watershed area in square miles.

a=a variable coefficient depending on the character of the surface of watershed.

I=the maximum average rainfall intensity throughout the entire watershed, expressed in inches per hour, during the time (t) in minutes, which is the time that it will take water under maximum rainfall conditions to concentrate at the outfall point and produce maximum discharge.

It is to be noted that the application of this formula to a part only of a watershed may give a result larger than that obtained under its application to the entire watershed. This is not inconsistent; but in that event the larger value of D_m, whichever it may be, will represent the desired maximum stream flow.

For use in the formula the value of I, the rain intensity which produces maximum discharge, is to be ascertained as already explained. The coefficient a which represents the effect of soil absorption must decrease as the time which it takes the water to reach the outfall point increases. For a small impervious area, $a=1$. These facts have suggested the following expression for a

$$a=\frac{60}{60+c\sqrt{t}} \tag{3}$$

The coefficient c in this equation is to be determined from the surface conditions of the watershed and the following values are tentatively suggested:

For impervious areas	$c=0.1$
For mountainous areas	$c=2.0$
For rolling country	$c=3.0$
For rolling and flat country (by interpolation)	$c=(3 \text{ to } 10)$
For flat country (ordinary soil)	$c=10.0$
For sandy regions	$c=50.0$

These values are intended to apply under ordinary conditions in temperate climates. They are probably too small in localities where rain falls on frozen ground or on snow.

TABLE 1.—*Values of the coefficient, a, computed from the formula,* $a=\frac{60}{60+c\sqrt{t}}$, *for use in the formula* $D_m=413\,aMI$

Critical time (t) minutes	Impervious areas $c=0.10$	Mountainous areas $c=2$	Rolling country $c=3$	Flat country $c=10$	Sandy regions $c=50$
5	1.00	0.93	0.90	0.73	0.35
10	1.00	.90	.87	.66	.27
20	.99	.87	.82	.57	.21
30	.99	.85	.79	.52	.18
60	.99	.79	.72	.44	.13
120	.98	.73	.65	.35	.099
180	.98	.69	.60	.31	.082
600	.96	.55	.45	.20	.047
1,440	.94	.44	.35	.14	.031
7,200	.88	.26	.19	.066	.014
14,400	.83	.20	.14	.048	.010
28,800	.78	.15	.11	.034	.007

Wherever dependable information relating to maximum storm water discharge exists for one or more streams which are comparable in the matter of surface characteristics with those of some stream for which the maximum possible discharge is to be ascertained, then the proper value of a or of c can be determined with fair dependability.

It remains to be stated that the simple relation existing between I and R permits of a substitution of R for I in the formula which may therefore be written in two forms:

1. When I—that is, the maximum intensity of rain on the watershed during the critical time—t, has been ascertained then as already noted:

$$D_m=413\,aMI$$

2. When R—the maximum rainfall (in inches) in one hour, is known then, because $I=\frac{R\sqrt{60}}{\sqrt{t}}$, there will be:

$$D_m=\frac{3,200\,aMR}{\sqrt{t}} \tag{4}$$

APPLICATION OF FORMULAS ILLUSTRATED

The application of these formulas can best be illustrated by a few examples:

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[2] Rainfall and Run-off Studies. Trans. Amer. Soc. of Civ. Engrs., Vol. LXXXV, p. 100.

Example I.—What is the maximum discharge of a river draining a mountain watershed of 1,900 square miles, in which the maximum amount of rain averaged for the entire watershed is 8 inches in 24 hours, and for which the critical time, t, is 12 hours or 720 minutes?

Here $c=2$ and therefore from equation (3)

$$a=\frac{60}{60+2\sqrt{720}}=0.53$$

This value of a could have been taken from the table in the mountain column by interpolation.

Equation (1) gives

$$C=\frac{8}{24}\times\sqrt{1,440}=12.6$$

and

$$I=\frac{12.6}{\sqrt{720}}=0.47 \text{ inches per hour.}$$

Therefore from (2)

$$D_m=413\times0.53\times1,900\times0.47=195,000 \text{ second-feet}$$

or; because

$$R=\frac{12.6}{\sqrt{60}}=1.63.$$

Therefore from (4)

$$D_m=\frac{3,200\times0.53\times1,900\times1.63}{\sqrt{720}}=195,000 \text{ second-feet.}$$

The conditions suggested in this example are comparable with those which prevail in the watershed of the American River above Folsom, Calif., which from a watershed of 1,900 square miles has probably discharged 180,000 to 200,000 second-feet at the peak of a flood period (1861–62).

Example II.—What is the maximum discharge of a river draining a watershed three-fourths mountainous and one-fourth rolling or foothill land, 9,000 square miles in area, in which the maximum amount of rain in two days over the entire watershed is 6 inches and for which $t=30$ hours or 1,800 minutes?

Here a will be found by interpolation between two columns of the table or

$$a=\tfrac{3}{4}\left(\frac{60}{60+2\sqrt{1,800}}\right)+\tfrac{1}{4}\left(\frac{60}{60+3\sqrt{1,800}}\right)=0.39.$$

Equation (1) gives

$$C=\frac{6}{48}\sqrt{2,880}=6.7$$

and

$$I=\frac{6.7}{\sqrt{1,800}}=0.158 \text{ inches per hour.}$$

Therefore from (2)

$$D_m=413\times0.39\times9,000\times0.158=229,000 \text{ second-feet.}$$

The conditions suggested in this example are comparable with those which prevail in the watershed of the Sacramento River, Calif., at Red Bluff, where the maximum recorded discharge has been about 250,000 second-feet.

Example III.—What is the maximum discharge of a river draining a watershed of 250,000 square miles, equal parts of which are mountains and rolling country. in which the greatest amount of rain in one month over the entire watershed is 6 inches and for which the critical time, t, is 20 days or 28,800 minutes?

Here a is again to be found by interpolation between the two columns of the table or:

$$a=1/2\left(\frac{60}{60+2\sqrt{28,800}}\right)+1/2\left(\frac{60}{60+3\sqrt{28\ 800}}\right)=0.13$$

Equation (1) gives:

$$C=\frac{6}{720}\sqrt{43,200}=1.73$$

and $$I=\frac{1.73}{\sqrt{28,800}}=0.0102$$

$$D_m=413\times0.13\times250,000\times0.0102=137,000 \text{ second-feet.}$$

This third illustration is typical of the conditions which prevail in the watershed of the Colorado River, which has an area of about 245,000 square miles (above Yuma) and from which the maximum run-off is at times about 0.6 second-foot per square mile.

When applied to small areas as in the case of studies for capacities of storm-water sewers, it will be found convenient to use the formula in its second form, equation (4), and to express area in acres instead of square miles as the unit. The formula will then appear in the form:

$$D_m=\frac{5a\ M'\ R}{\sqrt{t}} \quad\text{------------------------}\quad (5)$$

Here M' is the area of the watershed in acres, R the maximum rainfall in one hour and a the coefficient determined by surface characteristics as shown in the table.

Example IV.—To illustrate the application of the formula to a small area, let it be required to determine the maximum storm water flow from a combined urban and suburban area 5,000 acres in extent of which one-third is impervious and two-thirds rolling country, and which is so located that the critical time t is 90 minutes in a region for which the maximum rainfall in one hour is known to be 3 inches.

In this case, using the table, the value of a will be:

$$a=\tfrac{1}{3}\times0.99+\tfrac{2}{3}\times0.69=0.79.$$

Equation (5), therefore, gives

$$D_m=\frac{5\times0.79\times5,000\times3}{\sqrt{90}}=6,200 \text{ second-feet.}$$

This is at the rate of about 800 second-feet per square mile.

In using any formula for maximum stream flow it must be remembered that the value of R as determined from single station records is not the value of R for an area of considerable extent, because the area over which there is coincident maximum rainfall is limited. For the large area this value will be somewhat less than that deduced for the time t from single station records. The formula may, therefore, be assumed to include a moderate factor of safety when, as is customary, the single station records are used without any correction for the lapse of time between the arrival of a storm in one part of the watershed and its arrival in another.

The hope is ventured that engineers who are concerned with the determination of maximum storm-water flow from any watershed can be made to realize that no formula should be considered generally applicable unless it includes two variables, one based on maximum precipitation and the other on the surface characteristics of the watershed. In the case of all other formulas their limitations should be clearly set forth. They should not be called generally applicable.

DISCUSSION

By O. L. Grover, Bridge Engineer, United States Bureau of Public Roads

The need for a general waterway formula is apparent when one is confronted by several formulas and tables which are in use in different sections of the country. A formula is an aid in checking and comparing sizes of waterways but is generally not sufficiently close for the results to be used without the field data which should be obtained so far as possible. No formula should be used to supplant reliable field information. Frequently rough values are wanted in a hurry and later the checking of tentative sizes which have been determined in other ways is of great value. A means of comparing sizes is needed. It is believed such a means is at hand in the published charts on which the Myers formula has been platted. When applied on topographical maps so that drainage areas and topographical features are before one, great benefits should be had and for a minimum of labor and time.

Formulas having more factors to represent more or less independent variables may have justifiable advantage in finding proper sizes for storm-water sewers and other expensive structures, but the comparatively simple method suggested by Mr. Jarvis has much to recommend it for open country stream crossings where complete and accurate surveys are the exception rather than the rule.

By C. S. Jarvis, Associate Highway Engineer, United States Bureau of Public Roads

The formula proposed and successfully used by Mr. Grunsky is a good example of refined methods applied to a very elusive problem. The results are dependable in proportion to the basic knowledge, experience, and judgment of the manipulator.

Any expression that is flexible enough to cover the entire field of design is also capable of yielding erroneous results, because of the wide range in coefficients. The safest basis of control seems to be that which gives a direct comparison, such as the percentage scale affords.

If the variations due to extent of watershed are accounted for by employing some definite function of the drainage area, whether it be the square root or any other, then the influence of slope, vegetation, soil porosity precipitation, and other similar influences maywell be measured by changes in the coefficient, if the modified Myers formula is well founded.

To test the Myers scale for practical purposes, 12 problems were submitted to engineers of the highway and agricultural engineering branches of the United States Bureau of Public Roads; of the Southern Railway Co., the United States Army, and the Geological Survey.

This set of problems was solved in from one to two hours by reference to topographic maps, rainfall records, and the tabulated data published in connection with the paper entitled "Flood Flow Characteristics" published in the Proceedings of the American Society of Civil Engineers, December, 1924, to December, 1925. The results were as follows:

Stream and location	Drainage area	Ratings on the Myers scale determined independently by five engineers					Authoritative solutions	
		No. 1	No. 2	No. 3	No. 4	No. 5	Rating	Required waterway area [1]
	Sq. miles	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*Sq. ft.*
Bear Creek, near Denver, Colo	--------	15.0	14	12	14.0	14	------	--------
Hinkson Creek, Columbia, Mo	--------	15.0	12	13	17.0	14	------	--------
Santa Fe Creek, Santa Fe, N. Mex	--------	6.0	10	7	7.0	8	------	--------
Catoma Creek, Montgomery, Ala	285	10.1	12	15	10.0	10	12.0 8.4	[2] 2,030 [3] 1,420
White River, Seattle, Wash	424	14.0	14	12	15.0	12	12.3 8.8	[2] 2,530 [3] 1,810
Rock Creek, near Gettysburg, Pa	--------	21.0	30	25	25.0	25	------	--------
Marais des Cygnes (Osage), Ottawa, Kans	1,247	14.1	12	17	18.0	13	16.6	[2] 5,860
Rouge River, Detroit, Mich	--------	3.0	3	4	3.0	4	------	--------
Tallahatchie River, Batesville, Miss	1,595	12.0	9	11	12.0	10	9.7	3,870
Kootenai River, Bonner's Ferry, Idaho	13,000	12.3	12	10	11.0	12	11.0	[2] 12,600
Klamath River, near its mouth, northwest Calif	15,500	30.0	25	20	25.0	15	20.0 14.5	[2] 25,000 [3] 18,200
Rio Grande, Brownsville, Tex	230,000	10.0	6	6	7.5	8	7.9	[2] 38,000

[1] Derived from fundamental formula, $w=10\,p\sqrt{M}$, where M equals drainage area in square miles, w is the waterway area required and p the percentage rating.

[2] Rare.

[3] Frequent.

Only first approximations were attempted in each case, as the time was limited, but the results are in close enough agreement with each other and with the authoritative solutions, whenever available, to serve as a check.

The concensus of opinion among those who observed or participated in this test is that a single problem solved by other methods would ordinarily require nearly the same period of time as was used for the entire set with the Myers scale and method of approach.

Among the various rational and empirical waterway formulas which have met with favor, such as the one proposed by Mr. Grunsky, there seems to be the need of a common denominator or means of translation of results. Those who have tested the Myers scale have regarded it as a basis of comparison, where for example the flood intensities of drainage areas rating 10 per cent and 20 per cent respectively are shown in their correct relation even though the watersheds may be unequal in extent.

The adopted coefficient p for the modified Myers formula in any given case is a percentage representing the algebraic sum of all influences affecting run-off, and in this group rainfall, slope, and soil porosity are of prime importance.

Since the publication of the article "A General Formula for Waterways" in Public Roads, February, 1926, two diagrams have been prepared for use in determining the Myers scale rating of a stream where the drainage area and the discharge are known. In the diagram on page 87 the ordinate q represents the run-off per square mile while the diagram on page 88 employs Q, the total discharge in cubic feet per second corresponding to the observed flood stage. These diagrams are intended as an appendix to the former article.

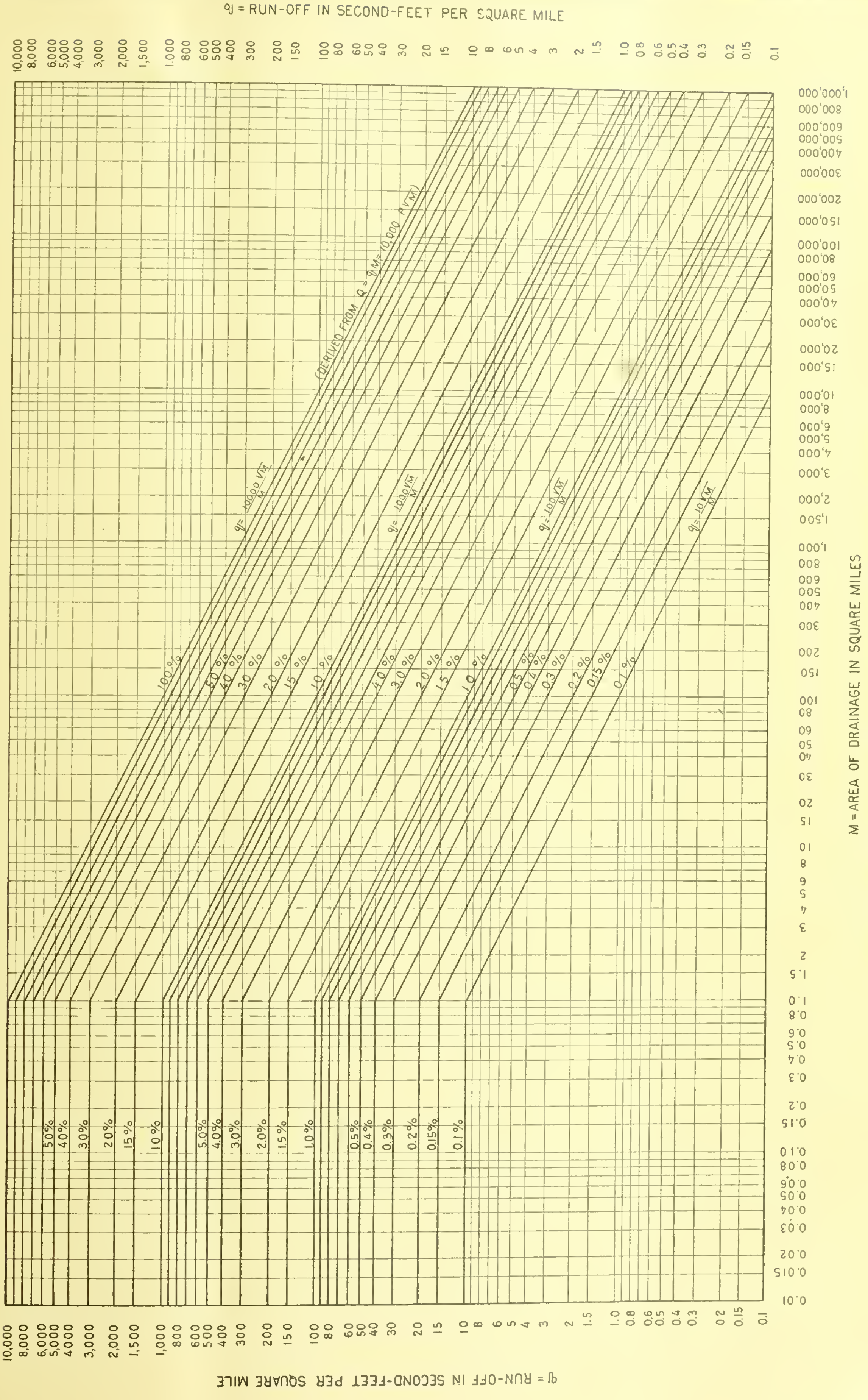

Diagram showing percentage rating on the Myers scale for any given yield per square mile

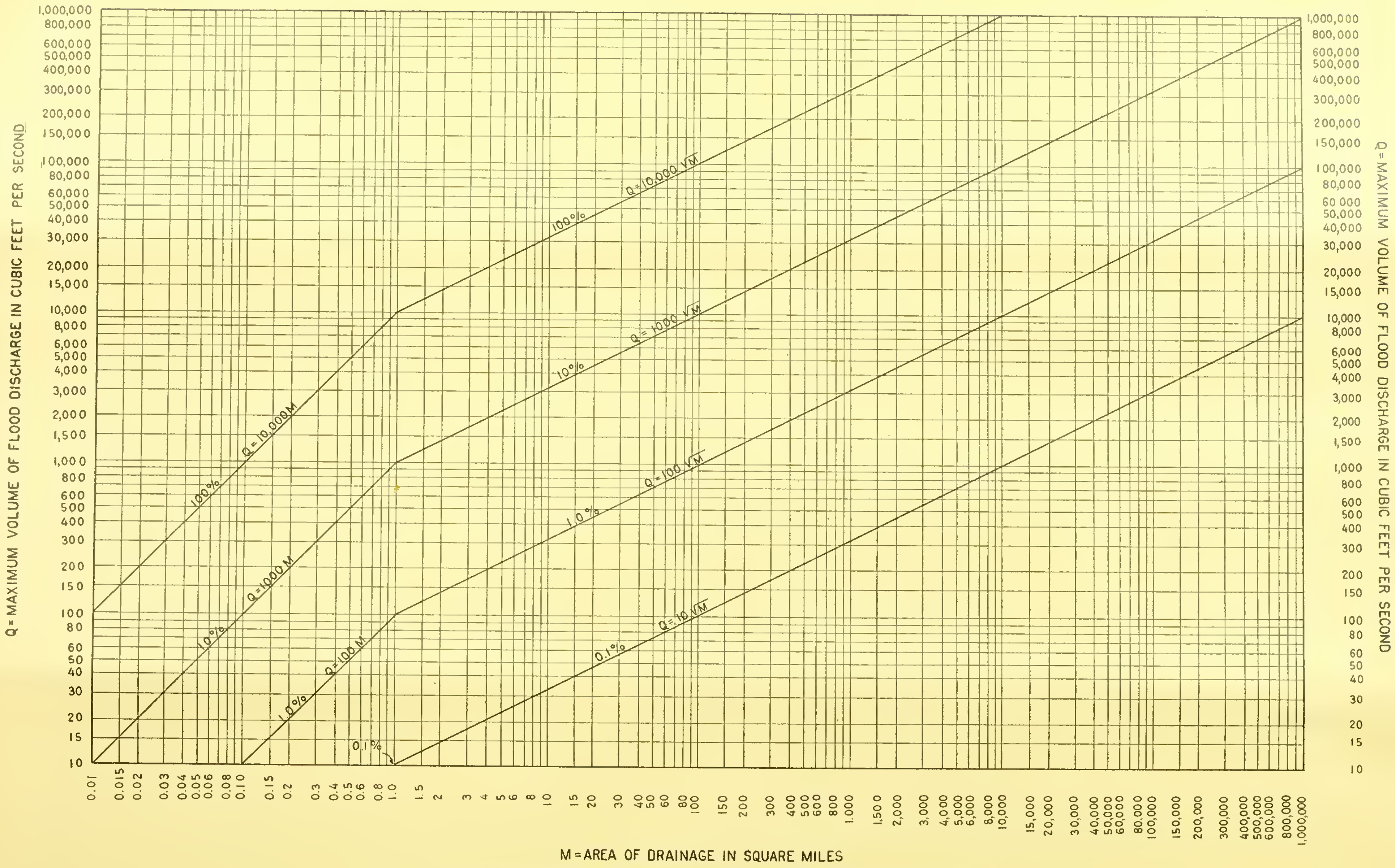

Diagram showing percentage rating on the Myers scale for any given flood discharge

SURVEY OF SOILS AND PAVEMENT CONDITIONS IN PROGRESS IN MICHIGAN

By V. R. BURTON, Engineer on Special Assignments, Michigan State Highway Department

A COMBINED pavement condition and soil survey was begun by the Michigan State Highway Department late in the fall of 1925. The work was discontinued because of extremely wet weather and subsequent freezing. At the time the operations were temporarily halted 108 miles of pavement-condition surveys had been made and 16 miles of general soil surveys were finished. The plans for 1926 include the survey of the remaining 2,000 miles on the State highway system. Whether the work will be accomplished depends to some extent upon the funds available. However, it is the intention to cover during the coming season at least a large variety of soils and pavement designs. Sufficient work has been done already to determine upon adequate methods for collecting and classifying the information; and the 1926 work will be begun with a clear understanding of the organization and methods necessary to complete the survey. Acknowledgment should be made of the material assistance secured from a study of the methods of the Pennsylvania State Highway Department pavement-condition survey.

The building of concrete pavements in Michigan has been going on at an increasing rate for the past 15 years, and it is probable that they will continue to be the predominating type of hard-surfaced road in the State. The designs of pavement slabs vary from the old 6–8–6 cross section with expansion joints at 30-foot intervals, through the 7 and 8 inch uniform thickness with or without expansion joints, to the existing 1924 sections with 10–8–10, 9–7–9, and 9–6–9 dimensions varied to suit the demands of traffic and the character of the subgrade. Various methods of subgrade treatment of the porous subbase type have been used in a more or less experimental way since 1921. Different designs of subdrainage and steel reinforcement have been in use throughout the same period.

The service records of the different types of design throughout the State should furnish some valuable information. It only remains to collect and classify these data for the purpose of deciding upon the relative effectiveness of the various methods of design. Information concerning the methods of construction, materials, and workmanship has been recorded by the State highway department especially on the more recent State-supervised jobs. The pavement survey was projected, therefore, to cover the major portion, if not all, of the pavements laid to date.

Lack of knowledge of the kinds and properties of subgrade soils is the most serious obstacle to a better understanding of the differences in the behavior of pavements laid under apparently identical conditions. It has been felt that this factor must be taken into consideration to make the pavement-condition survey of any great value. It seems to be generally believed that most sandy soils provide a good subgrade while clay soils as a rule are poor subgrade material. The intermediate type, however, with which we are more often concerned seems to be little understood or, worse still, not even recognized.

The decision has been made, therefore, to carry on a soil survey concurrently with the pavement survey for the purpose of determining the proper subgrade and pavement design suitable for a particular soil type under given climatic and traffic conditions. A discussion of this phase of the matter with the official in charge of the land economic survey being carried on in Michigan brought out the fact that about one-third of the State had been covered by some sort of soil survey by that agency and the State agricultural college, both in cooperation with the Bureau of Soils of the United States Department of Agriculture. The older soil maps are, for our purpose, of limited value only since the science of soil mapping and testing has been undergoing almost as rapid a change as that of highway engineering. In addition, the condition and properties of a subgrade soil must be determined in greater detail and from a different viewpoint than even the most recent agricultural soil maps will permit.

BUREAU OF SOILS SURVEYS REDUCE LABOR OF SUBGRADE STUDY

By adopting the Bureau of Soils classification of series and types and profiles it seems possible to diminish to a considerable extent the amount of laboratory testing. For instance the Fox sand is distributed quite generally over this State and differs only slightly throughout the whole area. The recognition of the fact that a given soil type in any particular climate and under similar conditions of drainage is bound to be much the same regardless of the location where it is found is a new idea to the highway engineer and not a very old one to the agricultural soil expert. The work of the Bureau of Soils together with our own State agencies has made soil mapping for highway purposes a practical possibility although this work was conceived and carried out almost exclusively for the uses of agriculture and forestry.[1]

About 100 different types of soil have been identified and mapped in Michigan up to the present time. While this may seem a large number it should be borne in mind that once a type is recognized and tested, very little future testing will be necessary. If every different soil encountered on a highway had to be tested in the laboratory the expense and labor involved would be prohibitive. The value of the Bureau of Soils maps is that they indicate the location of the various types, so that when the subgrade characteristics of all types have been determined the simple identification of the type will, in most instances, be sufficient to stamp the subgrade as good or bad.

The pavement condition survey party consists of one engineer familiar with construction and three assistants. Two of these assistants chain the pavement, marking it at every 20 feet and painting the station number every 100 feet. The third assistant

(Continued on page 92)

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[1] Methods of making soil surveys for highways using Bureau of Soils maps are discussed by A. C. Rose in PUBLIC ROADS, "Practical field tests for subgrade soils," v. 5, No. 6, August, 1924, "Field methods used in subgrade surveys," v. 6, No. 5, July, 1925, and "The present status of subgrade studies," v. 6 No. 7, September, 1925.

TESTS OF CONCRETE IN TENSION[1]

By **A. N. JOHNSON**, Dean, College of Engineering, University of Maryland

THE TESTS here described are the result of cooperative work between the University of Maryland, the State Roads Commission of Maryland, and the United States Bureau of Public Roads.

The purpose was to obtain more data as to the relative strength of concrete specimens in compression and tension, the tension specimens to have a cross-sectional area equal to those of the compression specimens.

To do this required special apparatus which was designed and made in the shops of the University of Maryland. Each tension specimen consists essentially of a cylindrical section, on each end of which inverted frustums of cones were made, so that the total length of the specimen was nearly 21 inches. The cylindrical portion of the specimen was the size of the cylinders used in the compression tests, 9 inches long by 4½ inches in diameter. The tension test specimens were cast in brass moulds which were split their entire length and bolted together. Before casting a specimen, the moulds were rubbed with an oily rag. The concrete was put in from one end and tamped with a rod. As the concrete was placed in the moulds, they were hit with a wooden mallet which resulted in a concrete free from small air pockets. The compression specimens were cast in steel moulds which were made by splitting seamless tubing along one element of the cylinders and holding the edges together by means of bolts attached to lugs. Compression specimens were tamped as the concrete was poured in one end and these moulds also were struck with a wooden mallet.

From a given batch of concrete, three specimens were made to be tested in compression and three in tension. No particular care was taken, however, that subsequent batches should have the same water ratio. The quartz sand used contained 4 to 5 per cent of silt, wet measure, and a trace of inorganic impurities, and had a fineness modulus of 3.28. The coarse aggregate was limestone not exceeding 1 inch in size, with a fineness modulus of 6.20.

APPARATUS FOR TENSION TEST

In the design of the apparatus to hold the tension specimen, special care was taken to insure that the geometric axis of the tension specimen should coincide with the line of force applied to the specimens by the testing machine. For this purpose there had been made a split aluminum cylinder of inside diameter and length equal to the diameter and length of the cylindrical portion of the tension specimen; and two conical, split, brass ends the smaller diameter of which was sufficient to permit them to be passed over the ends of the test specimen. In preparing the specimen for testing the aluminum cylinder was placed around the cylindrical portion and held in place by a wooden clamp. The brass ends, their two halves bolted together, were then passed over the conical ends of the specimen until they rested on flanges provided at the ends of the aluminum cylinder, using this cylinder to center them with reference to the axis of the specimen. There was thus left an annular space between the brass end pieces and the specimen, which was filled with molten rosin poured through holes in the top of the brass ends. As soon as both end pieces had been thus placed, the aluminum centering cylinder surrounding the center portion of the specimen was removed. Reference to Figure 1 will show the arrangement of the specimen and the apparatus. The diagram shows clearly how, by means of links and pins with rocker

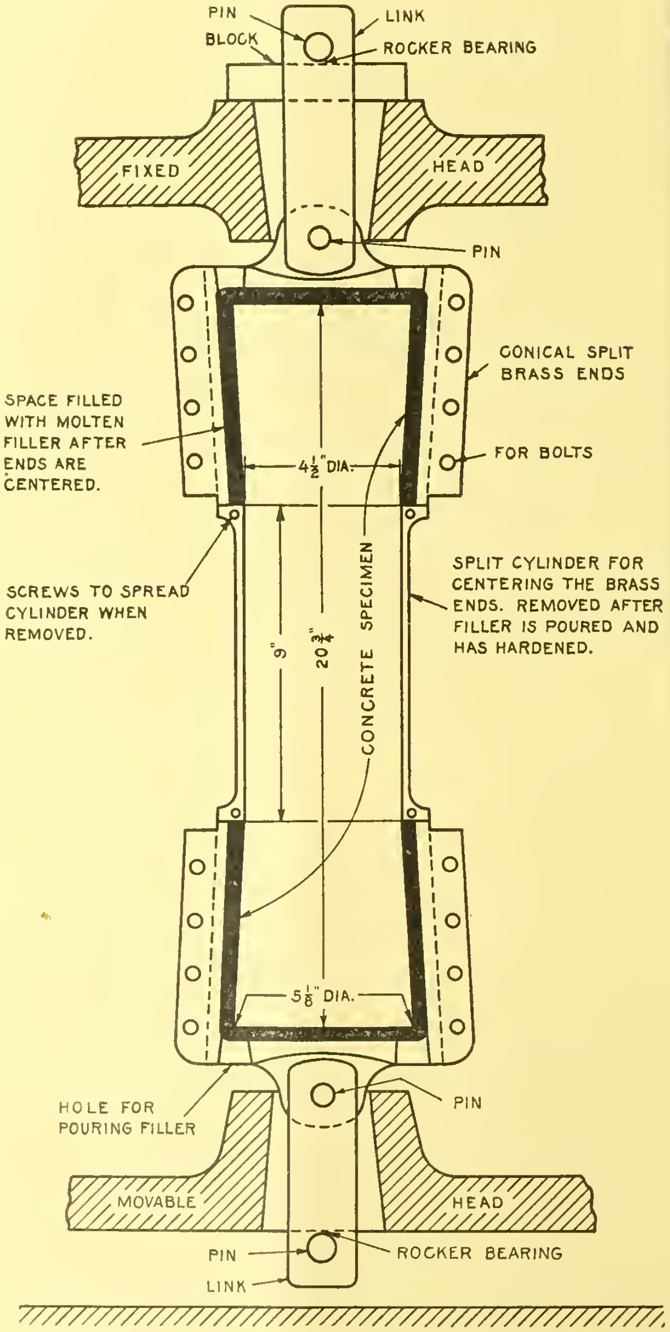
PIN
LINK
BLOCK
ROCKER BEARING
FIXED
HEAD
PIN
CONICAL SPLIT BRASS ENDS
SPACE FILLED WITH MOLTEN FILLER AFTER ENDS ARE CENTERED.
FOR BOLTS
4½" DIA.
SCREWS TO SPREAD CYLINDER WHEN REMOVED.
9"
20¾"
CONCRETE SPECIMEN
SPLIT CYLINDER FOR CENTERING THE BRASS ENDS. REMOVED AFTER FILLER IS POURED AND HAS HARDENED.
5⅛" DIA.
HOLE FOR POURING FILLER
PIN
MOVABLE
HEAD
PIN
ROCKER BEARING
LINK

FIG. 1.—Diagram showing the arrangement of the specimen and the apparatus for the tension test

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[1] This paper is substantially the same as a paper presented by the writer at the annual meeting of the American Society for Testing Materials, Atlantic City, N. J., June 22-25, 1926.

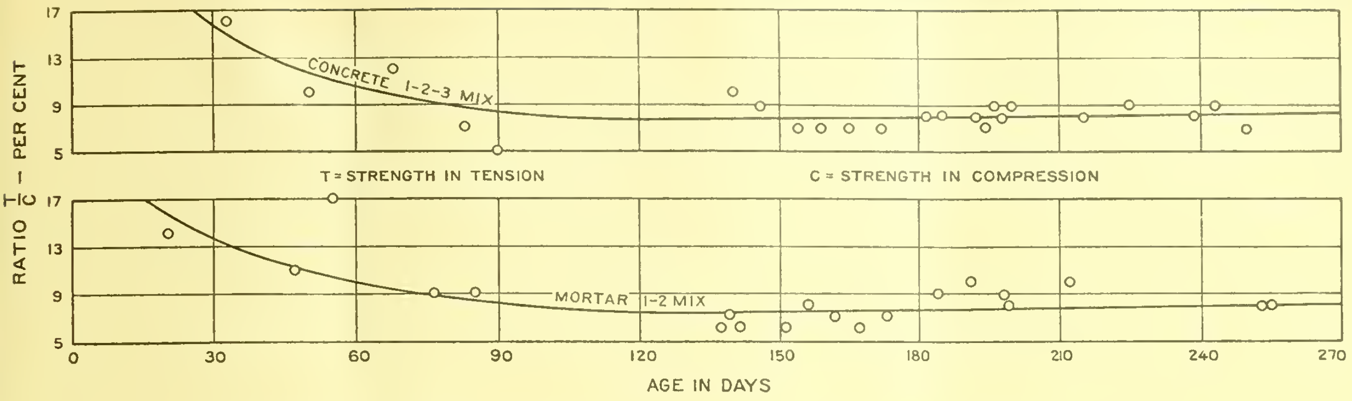

FIG. 2.—Diagram showing the ratio of tensile to compressive strength of concrete and mortar specimens of various ages

bearings, the specimen was mounted in the machine in such a manner that there was no twist or eccentric loading of the specimen during the application of the load.

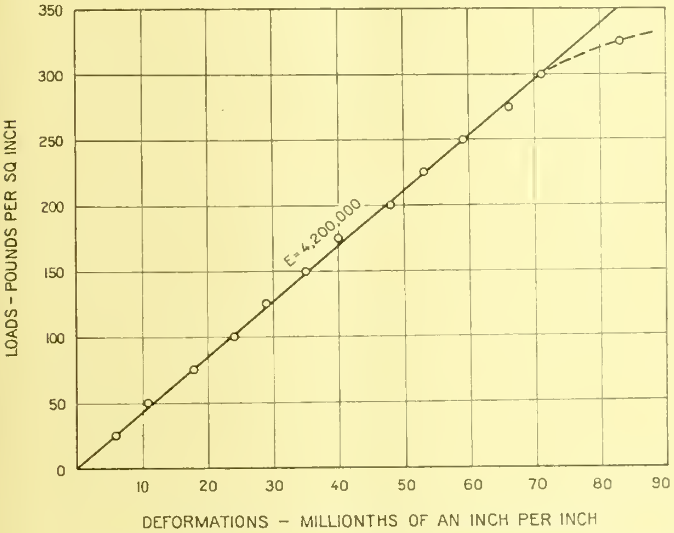

FIG. 3.—Relation of load to deformation of a 1:2 mortar specimen tested in tension

The compression tests were made with a spherical bearing block and usually a very thin cap of plaster was applied to the ends to insure flat bearings.

A 100,000 pound Riehle machine was used for these tests, the load being applied slowly by hand at a rate not exceeding 10,000 pounds per minute. In operation, the apparatus worked very satisfactorily.

The specimens were stored in damp sand for about two weeks and then removed to a room in the laboratory. They were tested at various ages up to 8 or 9 months.

The lack of any attempt to control the water ratio and the comparatively short period of curing accounts for the irregularity in the results when compared with the age of the specimens. As it was the intent primarily to study the ratio of tension to compression strengths, it was not necessary, in fact it was probably better, that there should be a variety in the specimens that would correspond with variations that might be expected in most commercial work.

Practically all the concrete tension specimens broke within or near the middle third of the cylindrical portion of the specimen, whereas a greater number of the mortar tension specimens broke near one clip or the other. In all instances, the breaks were nearly planes perpendicular to the axis of the specimen.

RESULTS OF THE TESTS

The results obtained are detailed in Tables 1 and 2. Table 1 shows the results for the mortar specimens, which were mixed 1 part cement to 2 parts sand. Table 2 shows the results obtained with concrete mixed 1 part cement, 2 parts sand, and 3 parts coarse aggregate. From the tables it will be noted, first, that the ratio of tensile to compressive strength is practically the same for the mortar specimens as for the concrete, and, second, that for specimens up to 90 days old the ratio in each case varies from about 15 per cent to about 8 per cent, and after this period, the ratio becomes practically constant at 8 per cent. It is apparent that during the first 60 to 90 days the increase in tensile strength is not as great as the increase in compressive strength, but beyond this time it appears that both are modified in practically the same ratio. The relation of these results is shown in Figure 2.

The agreement of the results seems so persistent, over such a wide variation in the character of the concrete that there is considerable definite evidence as to the ratio between the tensile strength and compression strength. Thus, in any given case the compression strength being determined, the tensile strength is made known with sufficient accuracy for most purposes.

During these experiments, a few measurements were made to determine the elastic curve of the concrete in tension. It is, perhaps, sufficient here to state a few of the results obtained.

TABLE. 1.—*Results of compression and tensile strength tests of cement mortar, 1:2 mix*

[Cross-sectional area of specimens, 16 square inches (approximately). Each result the average of tests of 3 specimens]

Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength	Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength
	Lbs. per sq. in.	*Lbs. per sq. in.*	*P. c.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	*P. c.*
20 days	2, 510	349	14	162 days	6, 123	399	7
47 days	3, 827	421	11	167 days	6, 750	431	6
55 days	2, 640	442	17	173 days	7, 133	466	7
76 days	4, 990	434	9	184 days	5, 790	525	9
85 days	4, 200	370	9	191 days	6, 340	630	10
137 days	6, 230	347	6	198 days	5, 420	494	9
139 days	7, 140	489	7	199 days	5, 873	496	8
141 days	7, 183	434	6	212 days	5, 613	541	10
151 days	7, 577	488	6	253 days	6, 663	547	8
156 days	7, 223	583	8	255 days	6, 183	505	8

TABLE 2.—*Results of compression and tensile strength tests of cement concrete, 1: 2: 3 mix*

[Cross-sectional area of specimens, 16 square inches (approximately). Each result the average of tests of 3 specimens]

Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength	Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength
	Lbs. per sq. in.	*Lbs. per sq. in.*	*P. c.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	*P. c.*
18 days	1, 297	------	------	181 days	1, 847	155	8
33 days	2, 160	349	16	185 days	4, 280	326	8
50 days	3, 197	335	10	192 days	3, 773	301	8
69 days	3, 043	360	12	194 days	4, 667	307	7
83 days	3, 007	220	7	196 days	2, 130	192	9
90 days	2, 503	126	5	197 days	2, 863	235	8
140 days	3, 353	334	10	200 days	2, 503	236	9
146 days	4, 160	369	9	215 days	4, 260	330	8
154 days	4, 175	293	7	225 days	3, 337	298	9
159 days	3, 310	244	7	239 days	3, 647	284	8
165 days	3, 473	257	7	243 days	2, 797	247	9
172 days	3, 023	221	7	249 days	3, 373	248	7

The deformations in tension were observed by means of a mirror extensometer attached in a manner similar to that described for observations of compression cylinders in a paper by the author to be found in the A. S. T. M. Proceedings, volume 24, 1924, part 2, page 1025.

As illustrative of some of the results obtained, Figure 3 shows the deformations obtained upon a 1: 2 mortar specimen, which broke at 340 pounds per square inch, and indicates a modulus of elasticity of 4,200,000 pounds per square inch. The values of *E* for the few specimens tested varied from 3,300,000 to over 5,000,000 for some specimens.

It is the expectation to carry on a more extended series of observations of the modulus of elasticity of concrete in tension, particularly with Lumnite cement.

(Continued from page 89)

accompanies the engineer following the chaining party. The engineer and this assistant walk along opposite sides of the pavement locating the cracks and sketching them on specially prepared cross-section paper. All surface defects, such as pitting, scaling, spalling, hair checks, etc., are noted on the sketch. About 4 miles of pavement is an average day's work for this party.

A soil surveyor skilled in soil mapping follows the pavement party and uses the painted stations to locate the boundaries of the various soil types. A strip of soil 200 feet wide is mapped. After the mapping is completed the soil surveyor returns over the same route and selects typical soil samples which are sent to the laboratory for routine tests. A sufficient number of samples are tested to determine the extreme upper and lower limits of variation within the type.

Both the pavement condition and the soil surveys are forwarded to the main office immediately upon completion of the field work and the information is transferred to the original plan which is on a scale of 100 feet to the inch. This sketch is easily readable when the work is done by a neat draftsman and the complete results may then be easily studied or compared. The type of soil, and depth of the roadway in the soil profile, the grades, cross section, surface and subdrainage are all noted and may be read at a glance. Surface data sheets of the pavement survey are prepared in such a way that separate sections of road may be compared slab by slab. The surface data sheets give the slab lengths, number and kind of cracks, the pavement classification based upon the length of the unbroken slab and the area of the pieces of broken slab, the amount of scaling, spalling, and hair checking, etc.

A study of one phase of the soil survey—the subsidence of fills in peat marshes—has been practically completed. The fills were cross-sectioned by borings so as to determine the actual shape of the floating fill. The classification of the peat strata was made in accordance with the system developed largely by Dr. Alfred P. Dachnowski, of the Bureau of Plant Industry of the United States Department of Agriculture. Testing of peats in accordance with the standard soil methods is impossible since oxidation takes place unless the sample is kept continually moist with the result that a totally different class of material remains. No screen analysis is practicable and shrinkage tests and bearing values must be obtained from the moist material without initial compression.

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Report of the Chief of the Bureau of Public Roads, 1925.

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Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.

Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MAY 31, 1926

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 11,644,803.06	$ 5,596,523.62	665.6	$ 5,469,336.08	$ 2,579,949.31	231.1				$ 3,309,784.21	Alabama
Arizona	9,580,133.43	6,016,119.94	613.8	1,230,100.53	748,233.76	102.3	1,537,643.55	1,043,510.75	95.0	$ 82,189.48	$ 50,225.99	12.2	2,759,158.56	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	4,643,529.91	2,081,104.37	261.5	4,940,580.79	2,370,447.28	306.2	536,670.32	236,160.00	37.4	1,537,910.62	Arkansas
California	22,346,175.99	10,719,249.51	894.8	4,513,257.39	2,114,444.58	150.9	11,170,314.24	5,518,926.90	312.8	604,506.02	245,734.28	7.6	3,474,459.63	California
Colorado	11,876,703.94	6,067,814.34	651.2	1,278,321.21	665,157.96	61.8	4,929,735.32	2,453,212.49	245.5	423,072.26	188,359.34	13.1	2,941,267.87	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	855,927.90	281,217.14	15.5	2,359,159.77	718,923.03	34.6	1,592,001.75	371,404.12	21.6	1,142,768.05	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1	636,492.48	285,474.95	17.2	1,115,536.81	470,577.90	28.2	496,458.35	204,710.90	12.2	17,103.50	Delaware
Florida	2,959,273.72	1,405,487.97	96.3	723,609.59	344,533.96	36.5	9,725,838.00	4,613,485.95	272.7				1,721,446.12	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	4,576,128.77	2,232,012.73	302.6	11,833,250.23	5,836,816.37	664.7	1,781,810.07	850,105.71	31.7	106,651.73	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	1,525,282.72	954,548.38	115.8	2,622,706.72	1,643,810.45	132.2	1,134,789.56	648,430.81	77.2	497,505.10	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	4,065,949.17	1,961,514.46	140.3	5,311,686.37	2,552,427.37	177.3				6,678,179.89	Illinois
Indiana	13,639,172.65	6,552,455.68	422.1	3,023,955.71	1,467,991.44	102.6	16,255,226.39	7,587,045.09	437.7	448,910.56	222,014.00	10.7	2,364,848.79	Indiana
Iowa	27,272,286.21	11,107,492.99	1,996.9	1,640,794.87	755,809.11	113.9	8,655,167.73	4,218,730.07	567.7	2,979,460.29	1,416,228.69	198.6	1,987,302.14	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	6,426,905.87	2,835,215.93	329.1	11,014,045.22	4,371,309.59	602.3	1,688,438.64	557,846.70	113.6	1,944,765.46	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	4,667,594.35	1,756,029.10	123.7	6,516,469.46	3,063,970.19	320.6	299,364.07	149,682.03	4.6	2,037,133.09	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,891,167.71	867,535.71	128.3	3,311,161.85	1,605,168.11	149.4				1,519,833.32	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	573,271.45	284,637.06	22.2	1,809,495.54	706,814.24	57.5	1,526,777.08	525,467.78	39.3	1,040,038.59	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,792,436.20	1,263,608.07	128.9	169,995.00	84,997.50	10.8	1,561,681.01	716,133.95	74.7	10,934.33	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	3,985,743.02	1,110,609.34	68.6	3,550,349.48	963,029.43	46.1	493,449.82	140,235.00	7.6	2,427,190.95	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,763,239.93	4,498,735.39	350.4	9,263,608.64	4,182,702.41	217.7	105,818.00	47,736.00	3.1	4,284,874.29	Michigan
Minnesota	30,415,685.89	12,738,642.04	2,721.2	5,271,152.99	2,284,974.52	397.0	9,821,971.75	2,975,500.00	629.6	464,657.41	34,000.00	15.0	1,558,663.44	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	3,910,793.14	1,954,397.32	278.9	7,120,916.68	3,529,463.68	368.2	2,168,639.90	992,466.87	111.6	662,987.40	Mississippi
Missouri	17,368,156.57	8,219,411.43	1,116.9	10,661,050.09	5,094,711.47	395.9	20,377,381.96	8,054,229.54	557.3	2,256,076.18	949,782.63	57.8	468,300.93	Missouri
Montana	10,156,600.41	6,317,523.15	921.6	1,244,383.40	1,015,942.74	133.3	1,523,431.87	1,008,165.06	120.4	836,221.14	456,127.69	97.7	5,627,126.36	Montana
Nebraska	9,306,374.36	4,389,523.50	1,570.6	2,169,810.65	1,056,279.01	195.0	12,109,329.60	5,998,839.67	1,215.4	1,541,463.70	769,987.45	164.0	2,421,205.37	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	2,640,729.82	2,042,634.81	181.5	3,136,209.75	2,698,004.08	328.5				966,276.33	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	826,870.74	391,223.20	29.5	582,611.55	274,125.52	17.7	241,941.63	117,707.65	8.2	400,208.76	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	4,227,678.25	1,236,143.72	68.5	6,746,521.08	2,515,017.44	31.3	240,608.13	45,765.00	3.1	849,813.85	New Jersey
New Mexico	8,717,999.16	4,914,070.61	1,081.3	3,568,872.54	2,326,079.86	329.6	1,456,200.02	966,613.62	116.3	112,357.05	70,735.50	5.6	2,694,886.41	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	12,579,727.57	4,913,674.20	314.7	32,079,571.58	9,123,202.24	577.7	7,890,953.00	1,765,990.00	114.3	6,013,252.03	New York
North Carolina	21,014,450.41	8,746,454.59	1,119.8	5,994,969.06	2,430,883.35	138.1	7,387,341.40	3,160,668.31	165.9	1,283,847.15	641,923.57	50.4	737,276.18	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1,917.5	1,455,792.12	748,801.58	27[illegible].6	4,382,686.58	2,236,028.14	570.9	3,840,966.05	1,876,966.93	493.3	617,931.86	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	6,059,391.44	2,107,214.63	171.8	9,432,894.33	3,653,206.34	293.8	3,927,570.42	1,381,899.48	103.2	3,344,481.62	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	6,496,136.84	3,025,537.08	278.7	3,949,958.99	1,877,399.11	140.8	482,026.22	235,088.31	52.3	1,248,872.16	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,325,783.62	1,256,645.21	111.3	3,462,508.87	1,913,780.24	159.5	519,886.14	303,949.91	22.2	262,607.01	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	17,824,416.03	5,169,421.37	325.6	25,155,981.30	7,060,697.63	512.9	3,107,544.09	1,032,120.89	67.5	1,854,517.14	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,360,119.89	439,140.97	21.9	1,531,802.80	427,155.00	28.5	45,705.65	13,050.00	0.9	668,534.94	Rhode Island
South Carolina	11,163,347.24	6,121,267.54	1,235.9	3,560,390.58	1,524,936.87	202.3	6,288,755.22	2,806,298.87	272.3	339,304.66	143,936.58	23.5	205,084.14	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1,447.9	4,801,631.44	2,329,698.81	657.5	3,582,932.31	1,722,393.53	570.4	482,685.86	262,655.48	104.2	862,163.18	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	7,584,709.49	3,419,113.70	265.4	7,900,200.62	3,625,744.50	265.1	616,648.03	259,926.86		1,243,726.17	Tennessee
Texas	64,120,970.83	21,057,940.12	3,907.1	14,578,815.45	6,146,695.22	942.7	18,773,855.28	8,311,683.32	961.1	3,243,754.71	1,459,817.79	86.0	3,630,394.55	Texas
Utah	6,259,159.41	3,818,836.91	423.1	1,543,632.88	979,511.08	90.0	2,149,723.47	1,573,821.69	187.5	385,929.62	297,447.91	27.4	1,149,161.41	Utah
Vermont	3,015,174.61	1,452,894.45	107.8	1,226,868.13	564,805.06	26.7	1,384,349.78	526,371.94	29.2	401,743.24	171,376.19	5.2	553,059.36	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,729,465.99	4,033,198.19	323.3	6,673,203.06	2,925,727.08	193.1	458,694.60	217,711.77	14.0	52,878.76	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,699,085.54	1,654,697.59	141.3	3,233,193.60	1,615,600.00	32.1	345,788.76	129,000.00	16.4	629,266.54	Washington
West Virginia	7,343,200.26	3,230,293.33	326.7	1,639,091.90	673,572.94	57.0	5,659,295.82	2,250,710.76	143.6	1,049,669.28	450,047.64	39.6	747,886.33	West Virginia
Wisconsin	21,807,140.91	8,910,640.62	1,451.7	2,994,612.24	1,446,735.85	137.4	5,633,325.43	2,774,909.19	251.5	1,734,700.38	838,574.00	94.5	3,458,955.34	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	2,087,000.03	1,282,218.38	151.5	2,553,262.00	1,610,353.02	193.7	149,528.34	95,995.00	20.5	838,610.93	Wyoming
Hawaii							1,050,897.83	312,635.18	15.9				787,517.82	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	211,521,493.75	93,653,725.79	9,869.6	336,701,611.92	144,123,599.13	13,860.9	53,924,308.52	21,584,526.42	2,463.6	86,358,802.66	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 77,031,615.32 Federal aid $ 33,660,927.65 Miles 3,018.2

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 5 ▽ JULY, 1926

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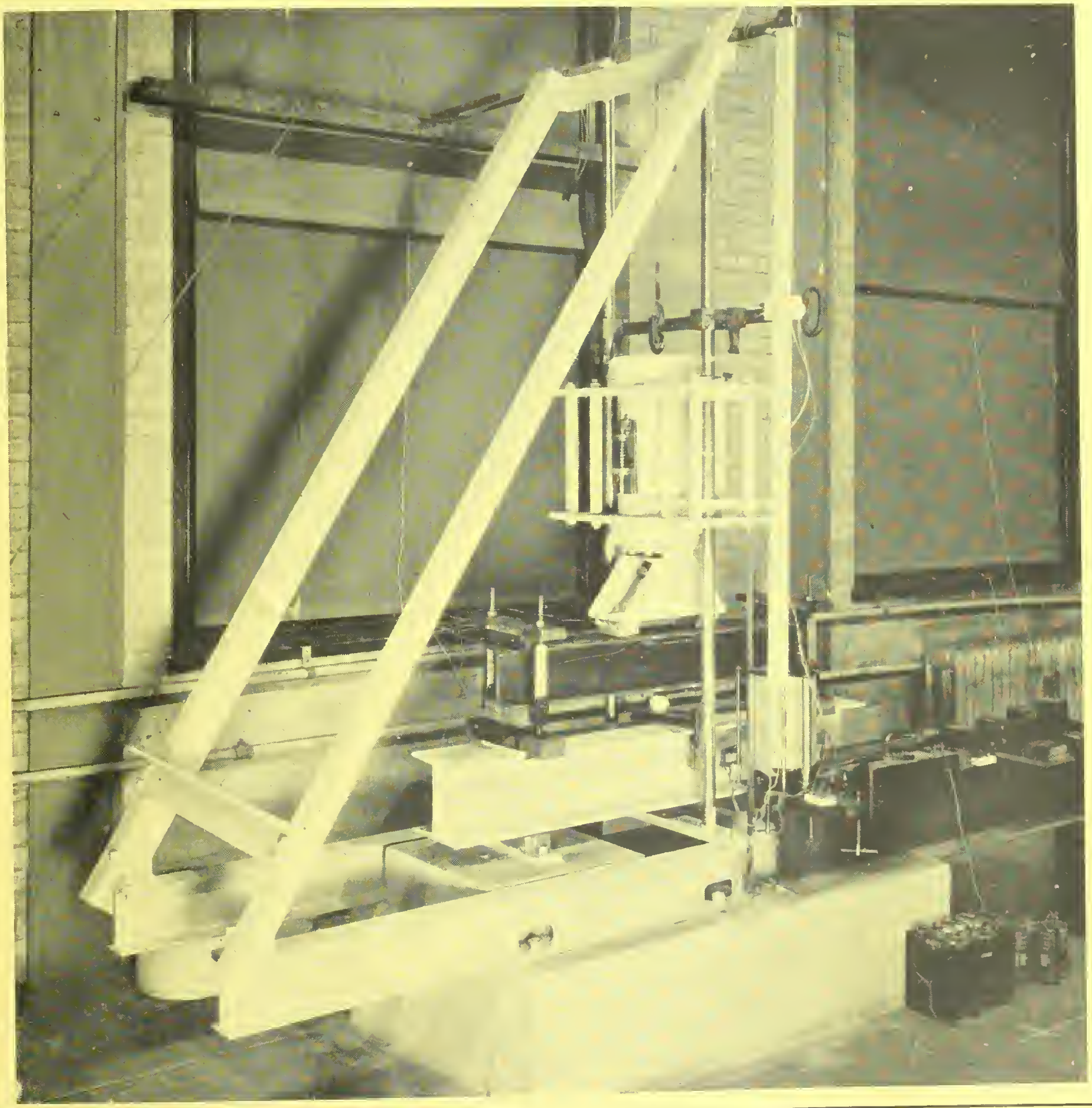

IMPACT MACHINE USED IN TESTS OF CONCRETE BEAMS

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1926

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 7, NO. 5 JULY, 1926 H. S. FAIRBANK, Editor

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STATIC AND IMPACT STRAINS IN CONCRETE

A COMPARISON OF THE MODULI OF RUPTURE OF BEAMS UNDER STATIC AND IMPACT LOADS

Reported by **J. T. THOMPSON**, Highway Research Specialist, Bureau of Public Roads [1]

IN THE summer of 1923, the Bureau of Public Roads brought to completion a series of impact tests on a large number of concrete slabs. These tests, which were conducted at the Arlington Experiment Farm, were subsequently reported in the issue of PUBLIC ROADS for April, 1924.[2] Among the striking features of this report were the exceptionally high unit fiber tensile deformations recorded. The average deformation for slabs of 1:1½:3 concrete was 0.000270 inch per inch; and since the average modulus of elasticity of the concrete was 4,480,000 pounds per square inch the indicated unit fiber stress in tension at elastic failure was 1,210 pounds per square inch. This modulus of rupture was recognized as being at least twice as great as the static modulus for the same concrete.

Apparently the discrepancy thus indicated was capable of but two explanations; either the strain gages used for measuring the unit deformations under impact were inaccurate, or the concrete subjected to impact was actually strained to a greater degree without rupture than is possible under static loads.

In order to throw some light, if possible, upon this question the Bureau of Public Roads and the Johns Hopkins University entered into a cooperative research project the aims of which were to study, in general, the action of plain concrete beams under impact forces and specifically to compare the static and impact moduli of rupture.

The economic importance of answering this question will be apparent when one considers that recent researches have evaluated the impact force delivered by motor vehicles to pavement slabs and shown it to be certainly double, and under some conditions several times the static weight of the vehicle. The natural tendency, in view of this information, is to increase pavement thickness; but if it is true that there coexists with the increase in force due to dynamic action a compensating possibility of stressing the pavement further without rupturing it, the increase in thickness might be unnecessary.

TESTS SHOW DURATION OF LOAD TO BE THE CRITICAL FACTOR

The tests prove conclusively that the graphic strain gauges used are free from inertia effects within the range of the use to which they were put; and the indications are that the deformations measured with similar instruments in the slab impact tests of the bureau are substantially correct as recorded by the instruments.

Doubt of the accuracy of the instrument being thus set at rest the experiment shows further that plain concrete beams subjected to static load have a considerably lower modulus of rupture than exactly similar beams subjected to successive impacts of increasing magnitude, caused by dropping a weight upon the beam from progressively increasing heights, when the weight is dropped but once from each height. Under these conditions, which are similar to those prevailing in the slab impact tests of the bureau, in which the weight was dropped only five times from each height, the beams resisted without failure impact forces considerably in excess of the static loads which caused failure. In other words, they were capable of greater elastic deformation under impacts applied as described than under ordinary static load applications.

When, on the other hand, the beams were subjected to repeated applications of impact of the same magnitude, the greatest impact they would successfully withstand was that which produced a tensile stress equal to about 55 per cent of the static modulus of rupture, indicating that the behavior of the concrete under impact is very similar to its behavior under static loads, as evidenced by static fatigue tests.

Below the fatigue limit it has been shown that equivalent loads, without regard to the manner of their application, whether static or impact, produce the same stress; and the indications are that the cause of the different behavior beyond that limit is to be found in the duration of the load application. This is evidenced by the fact that beams subjected to static loads for 10 seconds at a time showed the lowest modulus of rupture and impacts of an estimated duration of 0.015 second the highest with an intermediate value corresponding to impacts of an estimated duration of 0.030 second.

As an incidental result of the tests it has been shown that the variation in stress from the top to the bottom of a plain concrete beam may not be represented by a straight line but that the error involved in that assumption is small for ordinary safe working loads. It is also indicated that the impact force corresponding to a given height of fall is much greater when the fall is cushioned by worn solid rubber tires than when the cushioning medium is a new tire, thus accentuating the need for smooth pavement surfaces.

TESTS LOADS APPLIED IN THREE WAYS

Thirty-six plain concrete beams all of the same mix, with necessary control cylinders, were used in the tests. About one-third of the number were loaded at the third-points of a simply supported span by a 50,000-pound Olsen testing machine; with measurements of the load, center deflection and unit deformations of the concrete. These are referred to as the "static tests," although it should be borne in mind that "static" loads which are at their maximum intensity for a period of several seconds are in reality very slow "impact" loads.

Another group of the beams, one-third of the total number, was tested in impact by simply supporting the beams on suitable rests and allowing a heavy hammer, cushioned with full-thickness, unused, solid rubber tire segments [3] to fall from various heights, applying the the load to the beams at the third-points. The force of the blow, the deflection of the beam and the unit deformations of the concrete were measured.

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[1] Also associate professor of civil engineering at the Johns Hopkins University.

[2] "Impact Tests on Concrete Pavement Slabs," Leslie W. Teller, PUBLIC ROADS, vol. 5, No. 2, April, 1924.

[3] The tire segments used in the tests were generously supplied by Morgan & Wright (U. S. Tires), Detroit, Mich.

The remaining third were tested in impact in exactly the same manner as above except that the tire segments had part of the rubber removed to simulate a badly worn condition.

The reason for employing two conditions of impact—namely, the thick and the thin cushions was to introduce the time element—i. e., the duration of the force—the feeling being that the modulus of rupture, if different for static and impact conditions, would be found to be a function of the rapidity of application.

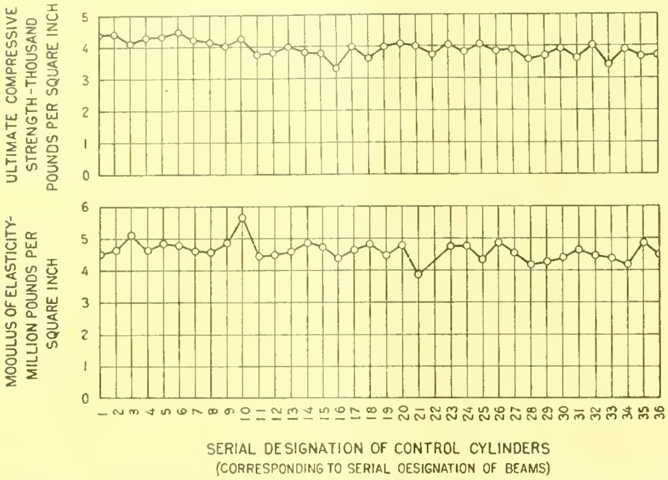

Fig. 1.—Data from Tests of Control Cylinders. Two Cylinders were Tested for Each Beam and the Plotted Points Represent the Average Values for the Two Cylinders

TEST BEAMS AND CONTROL CYLINDERS

The preliminary steps involved the preparation of test beams and control cylinders, the design and construction of an impact testing machine, and the selection and calibration of instruments of measurement. It was thought at the beginning that two beams a week were all that could possibly be tested and therefore only two a week were cast, since it was desirable that they all be of the same age when tested. This meant a lapse of 18 weeks between the casting of the first and thirty-sixth beam in which interim there would be abundant opportunity for differences in proportioning as well as in technique of mixing and placing unless great care were exercised.

Accordingly, enough material was procured for the whole job. The sand and gravel were dried bone-dry and stored, and all the cement was thoroughly mixed to eliminate possible differences between bags. The uniformity of the concrete was governed by controlling the water-cement ratio. In addition to this everything that went into the mixer was weighed and the fineness modulus of the aggregate was carefully checked for uniformity. The technique of mixing, placing, and curing was, as nearly as possible, identical in every case, and Figure 1 shows that a high degree of uniformity in the product was actually obtained.

The thirty-six 6 by 8 by 44 inch plain concrete beams of a mix closely approximating 1 : 1½ : 3[4] were cast; and for each beam two 6 by 12 inch control cylinders were made, making a total of 72 cylinders in all. The beams were cured under sand in a damp room, and were allowed to dry out at room temperature for one week prior to test. The cylinders were shipped to the Bureau of Public Roads, where they were cured in a similar manner and tested for crushing strength and modulus of elasticity on the test dates of the beams.

THE IMPACT TESTING MACHINE

The essential features sought in the design of the impact machine were: (1) A device that would drop a cushioned weight upon a simply supported beam from a predetermined height; (2) a method of support for the beam that would not restrain it and at the same time would prevent it from moving about when struck; and (3) a means of supporting deflection gauges that would preclude their movement relative to the beam.

The machine (fig. 2), which was built in the Johns Hokpins University shop, consists of a pile-driver-like arrangement equipped with a 265-pound hammer which can be dropped upon the beam from a variable height depending upon the setting of a trigger. The hammer is cushioned by two segments of a 3½-inch solid rubber tire mounted in such a way as to permit them to adjust themselves should there be a tendency for one to apply more load to the beam than the other. These apply the load to the beam at points about 14 inches apart, that is, at about the third points, the idea being to bring about a constant bending moment, and hence constant fiber stress, across the strain gauges located in the recesses shown at the center of the beam.

The test beam is supported at each end upon a cast-iron block, the lower side of which carries a half round running at right angles to the length of the beam. This half round fits into a V notch in a second cast-iron block, which is securely fastened to a heavy steel H beam. This "rocker" support permits the beam to deflect freely, while at the same time the angles shown on each side of the test beam in the center-line cross section (fig. 2) prevent it from moving laterally when struck. No tendency for the beam to move longitudinally was observed. It was at first thought necessary to hold the test beam down against rebound by means of shackles such as appear in the photograph on the cover, but these were found to restrain the beam and were therefore abandoned immediately in favor of the simpler and very satisfactory scheme of using the angles mentioned above.

The support for the deflection gauges is arranged for by inserting the 8-inch H beam between the block supports and the foundation. Since the H beam is supported upon knife rests directly under the reactions applied to it by the supports of the test beam it does not deflect. Only after careful verification of this fact by actual measurement was it used as a rest for the deflection gauges, however.

INSTRUMENTS OF MEASUREMENT

In both the static and impact tests the deformation of the concrete beams was measured by means of 12 graphic strain gauges arranged in slots cast in the concrete, 6 on each side, evenly spaced from top to bottom of the beam. These gauges, developed in the Bureau of Public Roads, are described in detail elsewhere.[5] They consist of a metal bar about 6 inches long and three-eights by one-quarter inch in cross section. At one end is an adjustable fixed point, at the other end a movable point which presses eccentrically against a small shaft. The shaft, thus made to rotate, carries a 4-inch light metal arm at the end of which is a small stylus which records its movement upon a small smoked-glass slide.

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[4] Acknowledgement is made of the valuable assistance rendered by Malcolm R. Gilpin, senior civil engineering student at the Johns Hopkins University, who volunteered his services in the preparation of these beams and cylinders.

[5] "Pocket Strain Gauge Gives Stresses in Concrete Roads," A. T. Goldbeck, Engineering News-Record, Mar. 29, 1923.

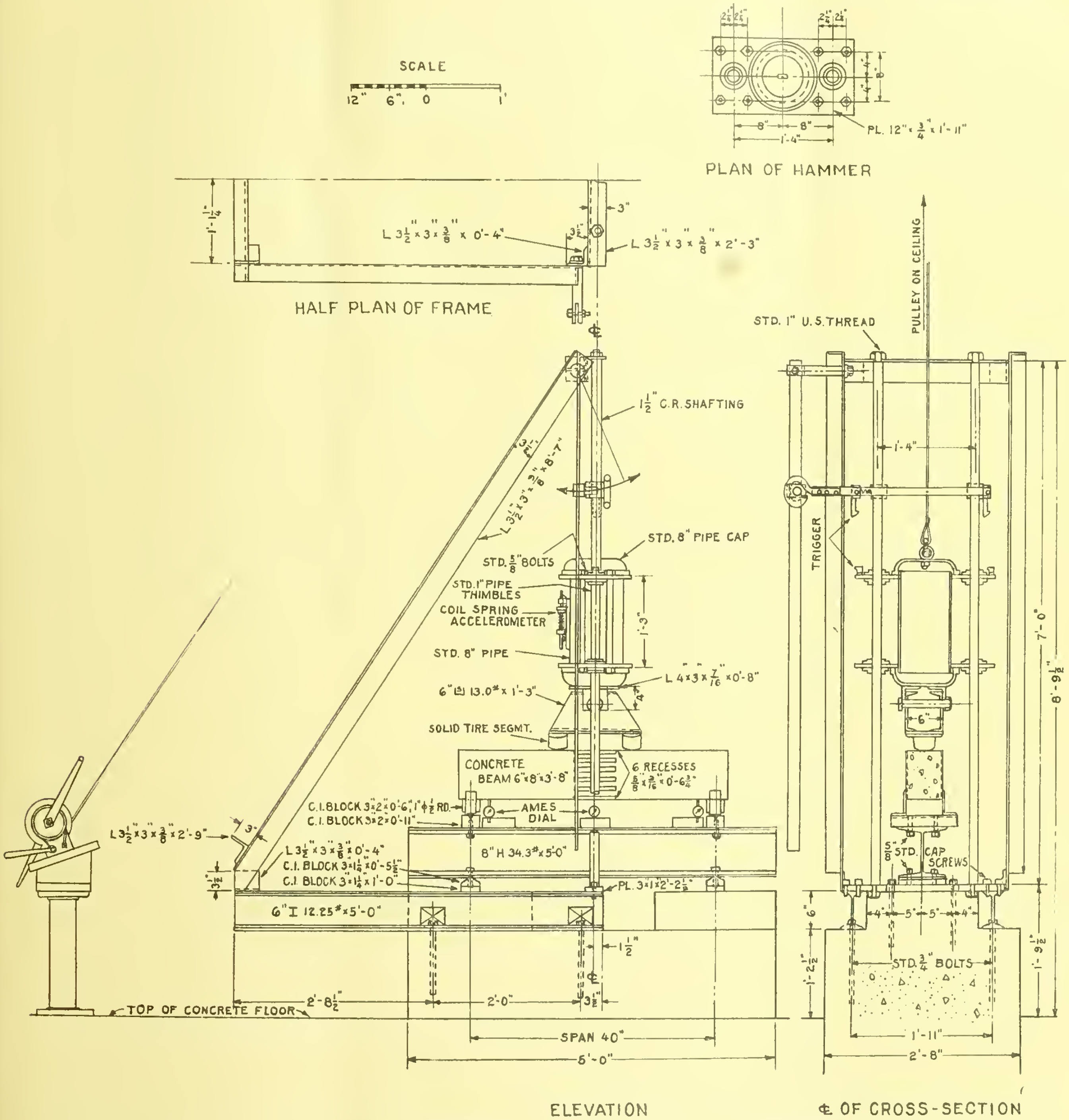

Fig. 2.—Details of the Impact Testing Machine Used for Determining Impact Stresses in Concrete

As previously intimated, it had been suspected since the development of these gauges that they were subject to inertia effects when used in impact—i.e., that they either lagged behind the true movement in tension or continued to record in compression after the impulse had ceased.

In order to settle this question the following method was adopted. A steel H beam was placed in the impact-testing machine and strain gauges were mounted on both tension and compression flanges between steel blocks securely fixed to the beam. The H section was so chosen that the product of its modulus of elasticity and its moment of inertia was the same as for the concrete test beams. This was done in order that the records of the strain gauges might be verified under conditions exactly similar to those to be imposed upon them in test.

The H beam was loaded statically by a screw jack and the strain-gauge records plotted against center deflections. The same deflections were then produced by impact and a similar curve was plotted. The curves of Figure 3, typical of several secured, show the results. It will be noted that the impact and static curves check very closely within the limits of deformation imposed upon them, which limits, incidentally, are about the same as those met with under test conditions.

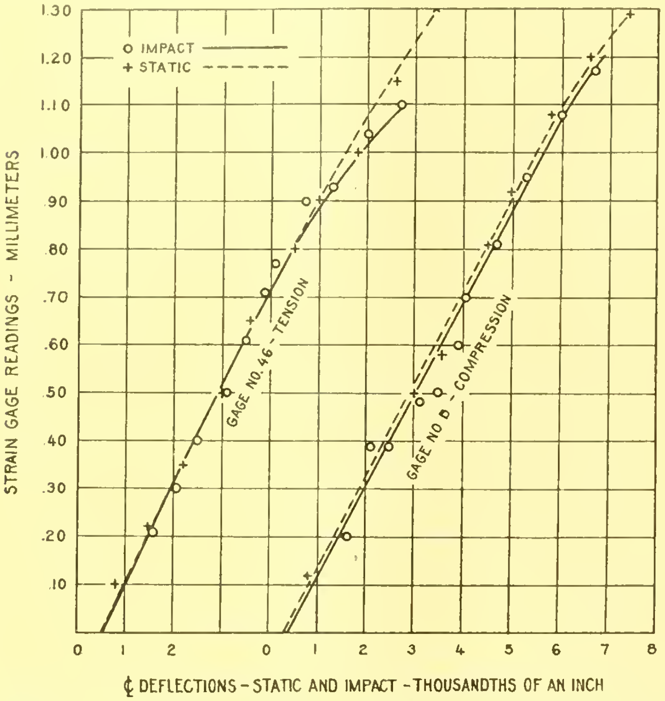

Fig. 3.—Comparison of Impact and Static Graphic Strain Gage Readings Showing Absence of Inertia Effect

This method of checking the instrument depends, of course, upon the premise that identical fiber deformations exist for a given deflection whether produced by static or impact force. This would seem to be true when it is considered that fiber deformation depends upon the radius of curvature of the bent beam. This is a well-known principle of mechanics. Also, assuming the beam bent to the arc of a circle, the measured deflection is merely the middle ordinate of a fixed chord. For every deflection (middle ordinate) there must be one and only one radius. From this it follows that fiber deformation varies directly with deflection.

The deflections of the beams in both static and impact tests were recorded by Ames dials reading direct to ten-thousandths of an inch.

In the impact tests the force of the blow was determined by the product of the mass of the hammer and its maximum deceleration as measured by a coil spring accelerometer the details of which are shown in Figure 4. The general type of this instrument is fairly common. It consists of a weighted plunger mounted vertically in two bearings, so that it is free to move downward against the action of a coil spring. The instrument was rigidly attached to the hammer of the impact machine and when the latter was decelerated upon striking the beam, the plunger of the accelerometer, accelerated downward with respect to the spring forced the latter to compress. The amount of this compression, which is a measure of the deceleration of the hammer, was recorded upon a smoked-glass plate by a stylus attached to the plunger.

Although crude in appearance and perhaps in scientific principle this instrument functioned perfectly for about a quarter of a million blows without once even so much as getting out of adjustment, thereby amply justifying its use in preference to more delicate types which offered themselves.

The calibration of the accelerometer was effected through the use of space-time curves. This method, which has been described elsewhere in detail,[6] consisted of the analysis of a curve, drawn by a stylus attached to the falling hammer, upon a sheet of paper wrapped on a drum rapidly rotating on a vertical shaft (see cover illustration). Concurrently a time curve was drawn on the paper by another stylus whose vibrations were governed by an electrically operated tuning fork of known period. Decelerations determined by analysis of the space-time curve were plotted against accelerometer records and a calibration curve thus obtained.

DESCRIPTION OF THE TESTS

Static tests.—These tests involved nothing out of the ordinary. A beam was supported, as shown in Figure 5, on the same supports as were subsequently used in the impact tests, the load being applied through a spherical head resting on the same tire segments as later were used in impact. The deflection gauge was carried by a yoke suspended from an ordinary "neutral-axis" bar.

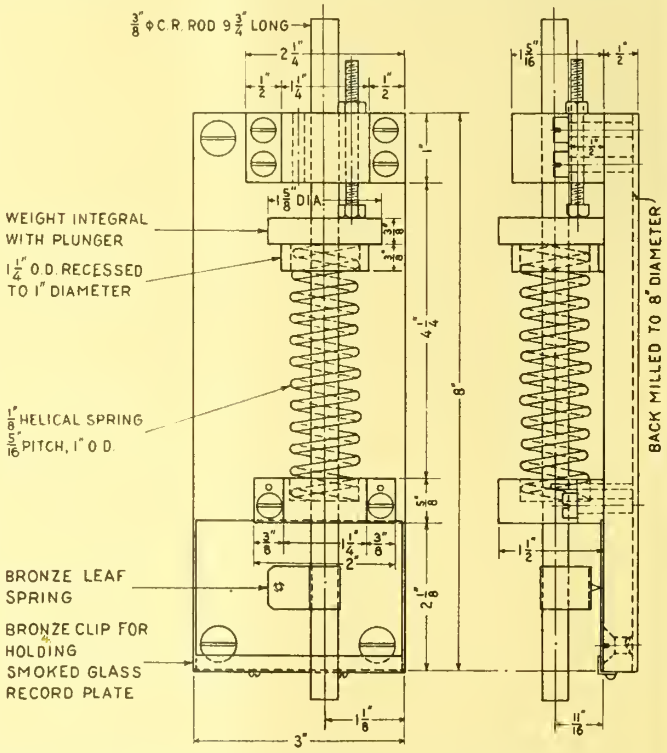
3/8" ϕ C.R. ROD 9 3/4" LONG
2 1/4"
1/2"
1 1/4"
1/2"
1 5/8" DIA.
WEIGHT INTEGRAL WITH PLUNGER
1 1/4" O.D. RECESSED TO 1" DIAMETER
1/8" HELICAL SPRING 5/16 PITCH, 1" O.D.
BRONZE LEAF SPRING
BRONZE CLIP FOR HOLDING SMOKED GLASS RECORD PLATE
1"
4 1/4"
8"
5/8"
2 1/8"
3/8"
1 1/2"
3/8"
1 1/8"
3"
1 5/16"
1/2"
1 1/2"
11/16"
BACK MILLED TO 8" DIAMETER

Fig. 4.—Details of Coil Spring Accelerometer Used in the Tests

The loads were applied at slow speed in increments of about 300 pounds, the load being released after each application at which time the strain gauge slides were moved up. The quantities recorded were loads, deflections, and unit deformations of the concrete.

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[6] "Accurate Accelerometers Developed by the Bureau of Public Roads," Leslie W. Teller, Public Roads, vol. 5, No. 10, December, 1924.

Impact tests.—Most of the beams tested under impact were hit but once from each height of drop, the height increasing progressively until the beam failed. The trigger was set so as to bring about the desired height of drop, the hammer raised by the winch until it was suspended from the trigger hooks, and then, after all gauges had been adjusted, the hammer was suddenly dropped by pulling the releasing lever. The deflections were recorded by "choked," ten-thousandths-type Ames dials, one dial being always used under the center of the beam, and frequently two others being used as near the supports as possible merely to detect any relative movement between the test beam and the H-beam. The strain-gauge slides as well as the smoked-glass plate on which the accelerometer recorded were removed at the end of the test and read.

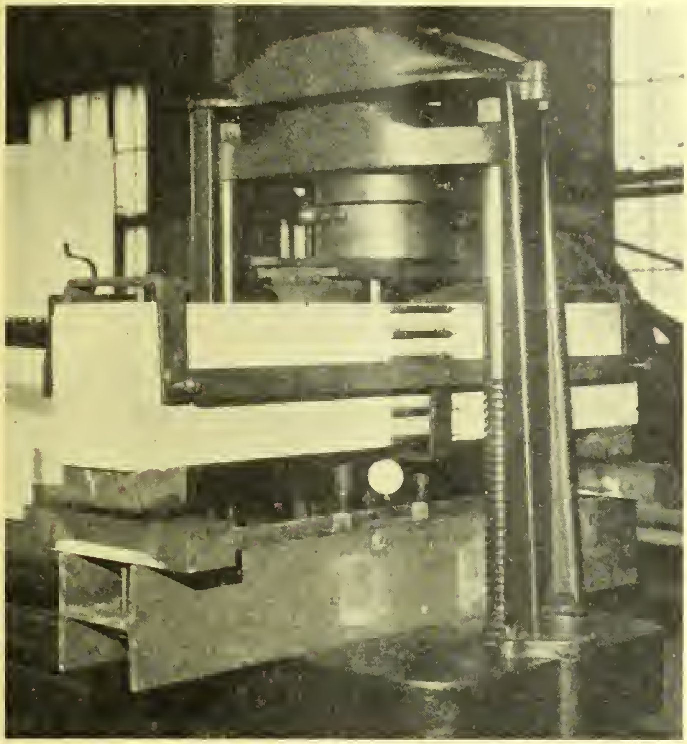

FIG. 5.—CONCRETE BEAM IN TEST MACHINE READY FOR STATIC TEST SHOWING ARRANGEMENT OF DEFLECTION GAGE AND GRAPHIC STRAIN GAGES

The effect of what might be termed "impact fatigue" was tried upon a few of the beams. In these tests the technique was similar to that just described except that a great many blows were struck from each height of drop. Readings were taken only when thought necessary, sometimes after a few blows, sometimes at the end of a series of several thousand.

The principal thing sought in each case was the modulus of rupture. In order to establish it, the unit deformations, measured at intervals above and below the center of the beam, were plotted to show the variation of strain. The fiber strains were then taken from these curves and plotted against load.

Before trying to ascertain from the strain-load curves where failure took place, some definition of failure was necessary. There seems to be considerable difference of opinion as to what constitutes failure in a concrete beam. Some experimenters do not assume a beam has failed until it breaks in half and falls from its support, others not until a visible crack has appeared, but the Bureau of Public Roads in practically all of its work has called that point failure at which the curve of strain plotted against load begins to change direction rapidly.

In the present instance, owing to the fact that the strain-load curves did not "break" sharply, it was very difficult to determine the strain at which this change of direction took place. Therefore it was decided to adopt that point at which the tangent to the curve would be a line with a natural slope of three-fourths (0.00015 inch per inch ÷ 2,000 pounds). This more or less arbitrary slope was chosen because it seemed to correspond in the majority of cases to loads creating a rise in the neutral axis of the beam and a sharper upward trend in the deflection-load curves. Figures 6 and 7, which are typical, show how the modulus of rupture was obtained; and data resulting from the static and "single-drop" impact tests are presented in Table 1.

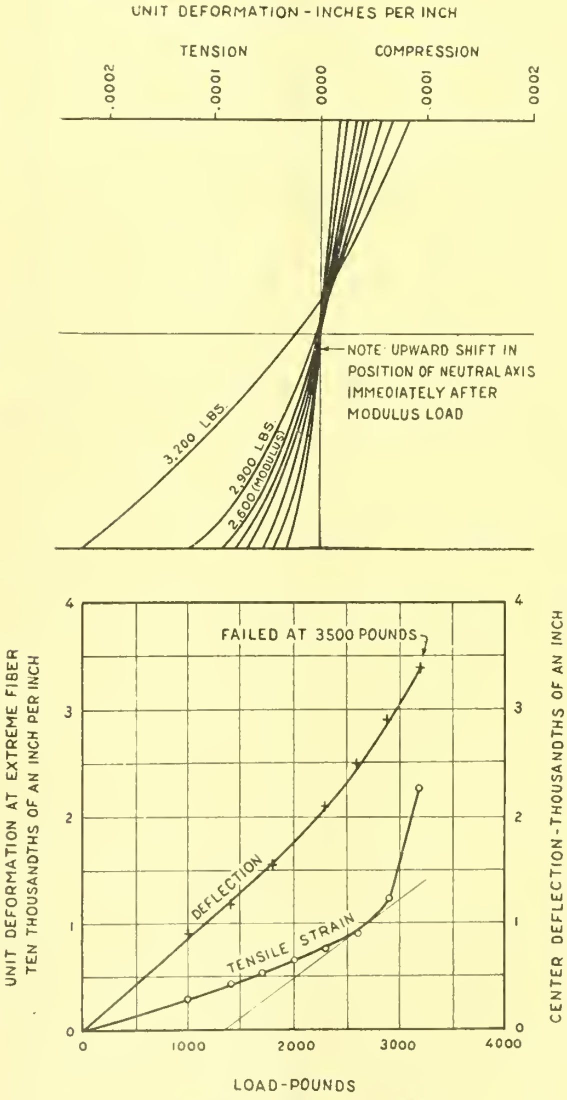

FIG. 6.—STATIC TEST OF BEAM A-4

HIGH IMPACT MODULI REDUCED BY REPETITION OF BLOWS

Where the mass was dropped but once from each height it appears that the average modulus of rupture under impact using the new cushion was 35 per cent higher than under static load, and 63 per cent higher when the worn cushion was used. (Figs. 8 and 9.)

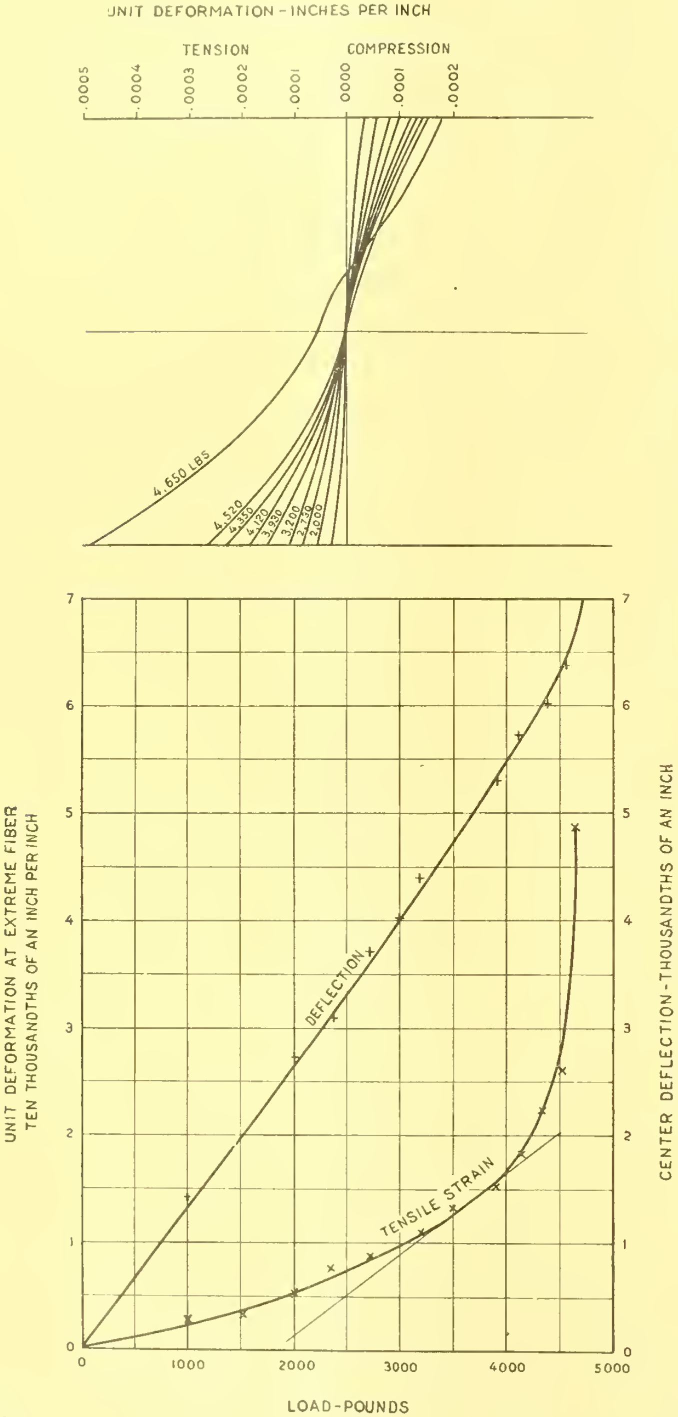

Fig. 7.—Impact Test of Beam A–11 with New Cushion

This conclusion is of little practical value, however, because of the influence upon modulus of rupture of such factors as duration of load and fatigue under repetition. In other words, later tests show that if the load application is repeated a number of times at each successive height it is possible to produce failure at much lower deformations than where the single-drop method is employed.

The explanation of this lies in the fact that it takes time for any material to fail. When the strain is increased by increments a critical stage is reached below which any number of repetitions is successfully resisted and above which one application of sufficiently long duration or a large number of short duration will cause failure, each application contributing toward complete and final destruction.

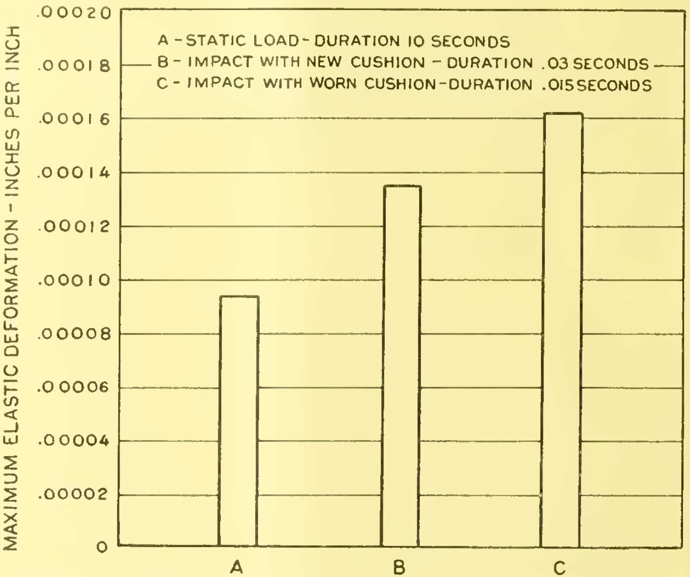

Fig. 8.—Variation of Unit Fiber Deformation for the Three Conditions of Test

Table 1.—*Data resulting from static and "single-drop" impact tests*

(Age of 1:1½:3 concrete 220 days)

Type of test	Beam No.	Ultimate breaking strength of control cylinders	Modulus of elasticity of control cylinders	Fiber deformation corresponding to modulus of rupture in single-drop tests
		Pounds per square inch	*Pounds per square inch*	*Inches per inch*
Static	1	4,410	4,500,000	0.000100
	2	4,430	4,600,000	.000105
	3	4,172	5,050,000	.000090
	4	4,360	4,600,000	.000095
	5	4,330	4,850,000	.000095
	32	4,045	4,450,000	.000082
	33	3,465	4,400,000	.000082
Average				.000093
Impact—new cushion	7	4,231	4,600,000	.000130
	8	4,175	4,550,000	.000125
	9	4,017	4,800,000	.000162
	10	4,226	5,650,000	.000150
	11	3,793	4,400,000	.000140
	26	3,864	4,900,000	.000100
	27	3,929	4,600,000	.000145
	28	3,564	4,200,000	.000115
	29	3,677	4,300,000	.000130
	30	3,944	4,350,000	.000140
	31	3,650	4,650,000	.000140
Average				.000134
Impact—worn cushion	13	4,062	4,600,000	.000180
	14	3,839	4,850,000	.000185
	16	3,304	4,350,000	.000140
	17	4,003	4,600,000	.000170
	19	3,942	4,500,000	.000130
	20	4,110	4,800,000	.000170
	21	4,045	3,850,000	.000190
	22	3,725	4,300,000	.000140
	23	4,062	4,750,000	.000150
Average				.000162

Accordingly, we should expect to find the static load-deformation curves swinging upward at a comparatively low modulus value when we consider that the time of application was about 10 seconds. The concrete was given a relatively long time in which to accomplish partial failure under each load beyond the critical stage referred to. No doubt had some load considerably below the modulus load been permitted to remain on the beam for a very much longer time than 10 seconds, the beam would have failed and we would have had a much lower value of the modulus of rupture. Or again had the static loads above the critical stage been repeated a great number of times the beam would have failed with accompanying low modulus.

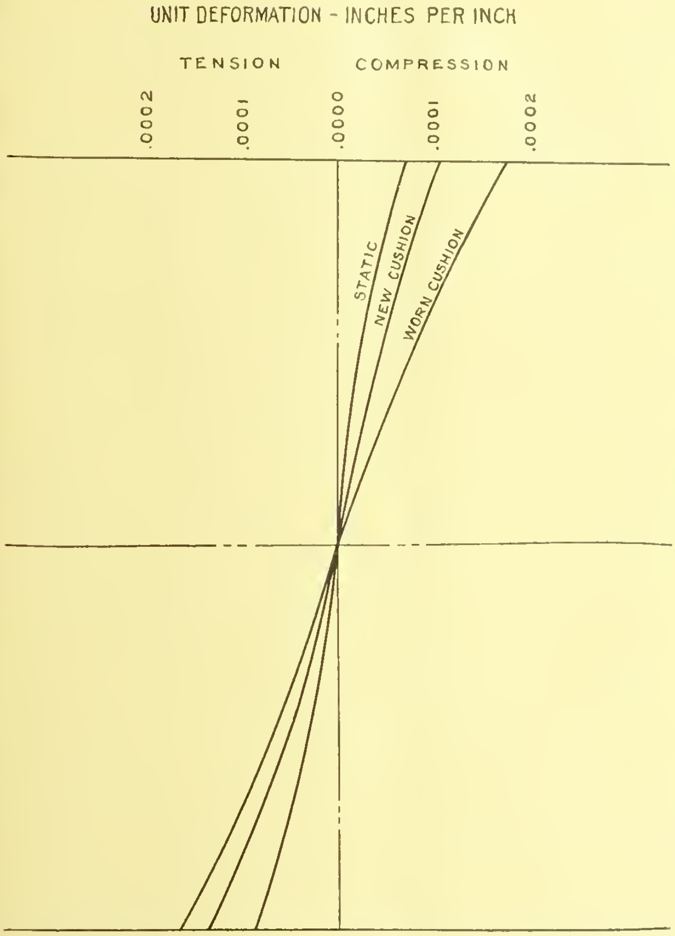

Fig. 9.—Diagram Showing Strain Distribution for the Three Conditions of Test at Modulus Loads

By the same argument the impact deformation-load curves should begin to change direction at a relatively high modulus value because the time of application in this case was only a few hundredths of a second and the concrete was given but little opportunity to fail under any blow. However beyond the critical stage had any application been repeated a great many times we ought to find the beam failing under these repetitions, and this is exactly what takes place in the impact-fatigue tests.

The composite strain-load curves (fig. 10) demonstrate this partial failure very well, although it should be noted that since they were obtained by averaging a great many results they are not particularly accurate, especially at the high values. The curves for the three conditions run along at about the same general slope (a given load causing the same strain regardless of how it is applied) until a value of strain (0.00005 inch per inch) is reached, beyond which the impact fatigue tests tell us we may expect things to happen. After this there is a rapid departure of the static curve from the other two due to the appreciably longer duration of the static loads. The "new cushion" curve, owing to its faster action (duration about 0.03 seconds) continues at the original slope for a while and then begins to swing upward while the "worn cushion" curve holds to the original slope longest because it is the fastest acting (about 0.015 seconds).

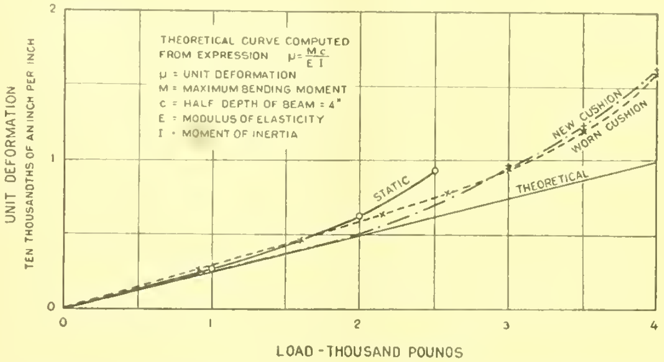

Fig. 10.—Composite Load-Deformation Curves Developed from All Tests

It is felt that the foregoing dicussion explains why such high moduli of rupture were encountered in the impact tests of slabs referred to at the beginning of this report.

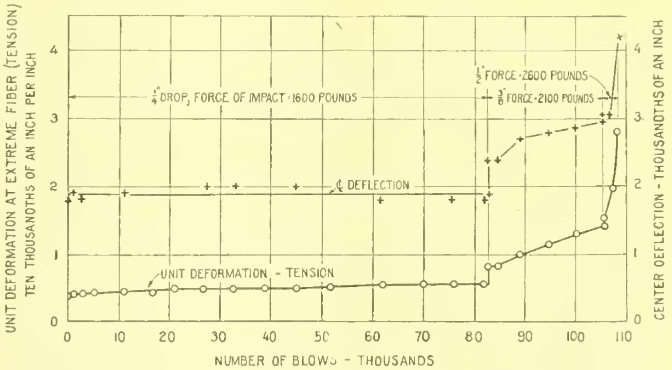

Fig. 11.—Impact Fatigue Test of Beam A-36. The Age at the Beginning of the Test was 277 Days; at the End, 307 Days

IMPACT FATIGUE TESTS

Toward the last of the series four beams were tested in impact fatigue, the rate of loading being approximately 60 blows per minute. Tests were carried on about three hours each day thereby allowing a 21-hour rest period between loadings. This method was employed in an attempt to show the effect of repetition of stress.

The results are shown by the curve (fig. 11) which is typical of all four impact fatigue tests in its essential features. It shows very clearly that for drops of one-quarter inch, producing a strain of 0.00005 inch per inch (corresponding unit fiber stress about 250 pounds per square inch) the beam can be subjected to a very large number of impacts without deleterious effects. As soon as this height is increased to three-eights inch, producing a strain of 0.00008 inch per inch slowly progressing disintegration sets in resulting in final failure.

Since the number of impacts successfully withstood at a strain of 0.00005 inch per inch, in this instance 83,000, is considered to be conservatively representative of any condition under which road slabs are used, the conclusion is reached that this is the upper limit of safe values to which concrete of this mix and age should be subjected.

It is interesting to note, in this connection, that the American Society for Municipal Improvements in a recently issued report of its committee on "Proposed Specifications for Portland Cement Concrete Pavements" recommends an allowable working stress of 250 pounds per square inch for 1:2:4 concrete. It also recommends that "in hot weather, the pavement shall be closed to traffic for at least fourteen (14) days, * * *." As has been pointed out, these tests show that where the pavement is to be subjected to impact fatigue, 250 pounds per square inch is the ultimate safe value for 1:1½:3 concrete, laboratory mixed and carefully cured for 300 days. Considering that concrete trebles its flexural tensile strength between 14 and 300 days,[7] it is interesting to speculate upon what might happen to concrete pavements built according to this specification.

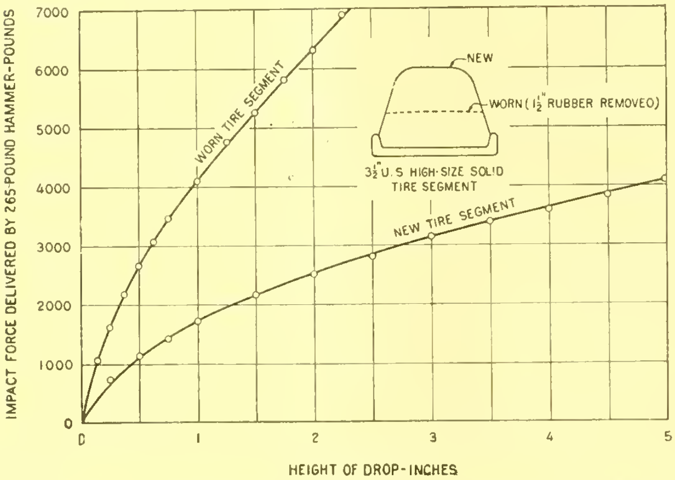

Fig. 12.—Curves Showing Effect of Tire Wear on Impact Force

It should be especially noted that the fatigue limit referred to above (strain 0.00005 inch per inch) is 54 per cent of the modulus of rupture obtained statically (strain 0.000093 inch per inch). It may therefore be concluded that under impact loading the same fatigue-limit ratio is obtained as has been shown to exist under static load conditions by the latest fatigue tests.[8]

An inspection of the strain-distribution diagrams (figs. 6 and 7) reveals very clearly the distortion of planes under bending. However, for low values of strain no very considerable error is made in the common assumption that they are not distorted.

The well-known increase in impact force produced by worn as against new tires is clearly demonstrated in the curves. (Fig. 12.) This shows the great disadvantage of using worn tires or, since the use of worn tires can hardly be controlled, the necessity of smoother pavements. The ratio of force produced by the worn tire to that caused by the new tire is about 2.5 throughout the range of comparison.

TRANSPORTATION SURVEYS BEGUN IN NEW HAMPSHIRE AND VERMONT

Highway transportation surveys covering the entire States of New Hampshire and Vermont were begun on July 16 under cooperative agreements between the Bureau of Public Roads and the two State highway departments.

As planned, the surveys are to run for three months, with 13 recording parties taking data twice during the period at each of 143 stations in New Hampshire and 12 parties recording the traffic at 130 stations in Vermont.

In general the same methods will be used as in the Connecticut and Cook County surveys, reports of which have been published, and the Pennsylvania and Ohio surveys which are now being analyzed. For passenger vehicles the data will show the number of vehicles passing the station during each hour, the State in which they are licensed, the number of passengers they carry, whether the purpose of travel is business or pleasure, whether passengers are from city or farm, whether the travel is an extended tour or a short trip, the origin, destination, length of trip, and the number of miles of travel within the State.

For trucks the data will show also the number passing each hour, the State of registration, capacity, origin, and destination, with information as to the character of operation of the consignor and consignee, the total distance of travel and mileage within the State, the commodity transported, the type of trucking (for hire, contract hauling, etc.), and the situs of ownership. In addition to this information there will be recorded at special weight stations the make of truck, type of tires, and weights on front and rear axles.

Analysis of this information will make it possible to prepare maps showing the present volume and distribution of passenger vehicle and truck traffic on the highways of each of the States, from which the State highways will be classified as industrial, high, medium, or low type routes, taking into account motor-truck capacities and loads. A forecast of traffic for several years ahead will be made and a definite program of highway construction worked out. Special attention is to be given to an analysis of the relative traffic importance of the township roads with respect to the primary and secondary systems.

Economic data to be collected will include information concerning the tonnage shipped by motor truck, marketing methods, zones of truck operation, and general trucking practice.

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[7] "Fatigue of Concrete," H. F. Clemmer, Proc. A. S. T. M., vol. 22, Part II, 1922, p. 415.

[8] "Fatigue of Concrete," W. K. Hatt, Proc. Fourth Annual Meeting, Highway Research Board, National Research Council, December 1924. Professor Hatt determined the limit to which concrete might be stressed an indefinite number of times as 55 per cent of the stress at which it failed under progressive static loading.

EFFECT OF SIZE OF BRICK ON RATTLER LOSS[1]

Reported by F. H. JACKSON, Engineer of Tests, United States Bureau of Public Roads

IT HAS long been recognized that for brick of equal quality but differing in size the comparative rattler losses are not directly proportional to the differences in weight. Therefore, any system of rating based on the percentage of loss by weight, irrespective of the difference in size, is incorrect unless a correction is introduced covering this feature or unless independent standards are set up for each size separately.

In the very elaborate series of tests presented by Blair and Orton before the American Society for Testing materials in 1911,[2] upon which our present standard method of test is largely based, no mention is made of the effect of size on rattler loss. This was probably due to the fact that at that time the bulk of the paving brick manufactured were of the so-called "standard block" size—that is, about 3 to 3½ inches in width, 4 inches in depth, and 8 to 9 inches in length. Professor Talbot, however, in his paper on "Qualities of High Grade Paving Brick," published in Bulletin 9 of the Illinois State Geological Survey,[3] states that, although he has not studied the effect of the size of brick on the rattler loss, "it is established that the brick size will sustain a greater loss than the block size of the same grade and quality. * * * The amount of this difference depends upon various conditions, but with good material the brick sizes may be expected to lose, say, 3 per cent more than the block sizes."

It has only been within comparatively recent years that the thinner brick, such as those made to lay to a depth of 3 inches and 2½ inches, have come into general use. Efforts have also been made from time to time to introduce certain odd sizes, such as the 3 by 3½ by 8½ inch, and others in which the length varied slightly. The number of sizes of brick in use, however, has been greatly curtailed within the last three years through the efforts of the permanent committee on Simplification of Variety and Standards of Paving Brick of the Department of Commerce. At the present time this committee recognizes two sizes of brick, as follows: 3½ by 4 by 8½ inches and 3 by 4 by 8½ inches. In addition to these two sizes, the 2½ by 4 by 8½ inch size is coming into rather general use, so that it will in all probability in the near future be included in the series of recognized sizes.

Recognizing the injustice of specifying the same percentage of wear for both 3-inch and 3½-inch brick, many paving engineers, when they began using the thinner brick for construction, adopted the practice of inserting certain arbitrary correction factors in their specifications so as to bring these sizes into line with the requirements for the so-called "standard block" size. So far as the writer is aware, however, none of these correction factors was based upon extensive test data. As a rule, they were the result of theoretical consideration.

In view of the fact that committee C–3 of the American Society for Testing Materials has undertaken to rearrange the standard specifications and methods of test for paving brick, C 7–15, in order to bring it into conformity with the society's present standards as to form, it seemed an excellent opportunity to investigate this relationship experimentally with a view to furnishing a table of correction factors which might be inserted in the standard.

TESTS MADE ON FIVE SIZES OF BRICK

Fortunately, a rather unusual opportunity existed for obtaining such data. The Bureau of Public Roads has had under way for the last several months an investigation to determine the relation between the depth of the paving brick wearing course and the resistance of the pavement to the action of traffic. This investigation is being conducted by the bureau in cooperation with the National Paving Brick Manufacturers Association, which furnished a large quantity of brick for this purpose. These brick were all from the same plant and were manufactured as a special lot so as to be as nearly uniform in quality as possible. They were of the plain wire-cut type, and the average rattler loss on the 3½ by 4 by 8½ inch was about 17 per cent. Five sizes of brick were furnished, all of the same width and length but varying in depth from 2 to 4 inches by ½-inch steps.

It was decided, therefore, in addition to the major investigation, the results of which will be reported later, to make an incidental study of the effect of size on rattler loss, using the five sizes of brick on hand. For this purpose 10 standard rattler tests were made on each size of brick, making 50 tests in all. Every precaution was taken to keep the standard rattler calibrated, and every detail as called for in standard procedure was followed. The results for the first series of rattler tests are given in Table 1. Upon studying the results it became immediately apparent that there was no constant relation between the average percentage of loss and the size of the brick. This at once suggested the possibility that the various sizes of brick might not all be of the same quality. In order to throw light upon this important point, a number of brick of each size were subjected to a special hardness test, using the Dorry hardness machine for testing rock. One-inch cores were drilled with a diamond drill from the center of each brick, and subjected to the abrasive action of quartz sand fed upon a revolving steel disk upon which the brick core was held under a standard pressure. The loss in weight of the specimen at 2,500 revolutions of the disk was considered to be a true measure of its hardness.

Realizing that hardness is not the only quality of a paving brick which affects the rattler loss, tests for crushing strength and transverse strength were also made. The results of these tests, together with the results of the hardness tests, are plotted in Figure 1. Crushing strengths were determined on half brick, tested on edge, five tests of each size being made. The brick were bedded in plaster of Paris before testing. Tests

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[1] A paper presented by the writer at the annual meeting of the American Society for Testing Materials, June 22 to 25, 1926, Atlantic City, N. J.

[2] "A Study of the Rattler Test for Paving Brick," M. W. Blair and Edward Orton, jr., Proc. A. S. T. M., Vol. XI, 1911, p. 776.

[3] Illinois State Geological Survey Bul. 9, "Paving Brick and Paving Brick Clays of Illinois."

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for transverse strength were made in two ways, (1) by using a special form of equalizer apparatus developed at the Bureau of Standards, and (2), by the use of the A. S. T. M. standard apparatus somewhat modified by the Bureau of Public Roads. Details of the results of the transverse tests, discussed from the standpoint of comparative methods of testing, will be published elsewhere. For the purpose of this discussion, the results of the tests by both methods were averaged. Each average is the result of 10 tests.

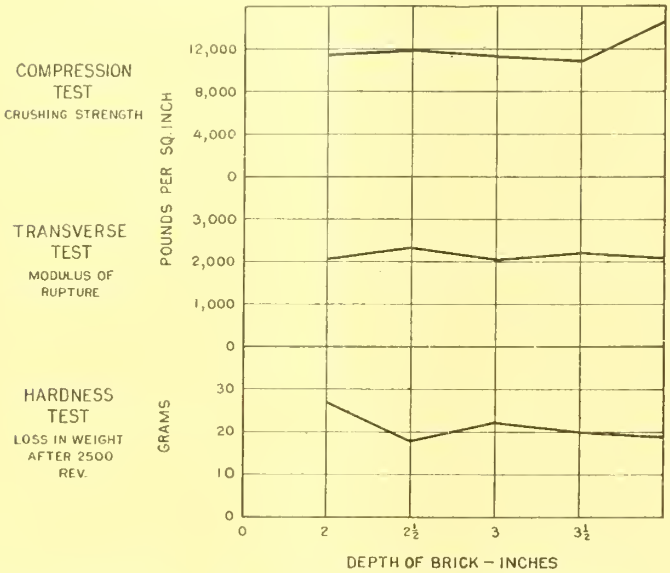

FIG. 1.—RESULTS OF HARDNESS TEST, TRANSVERSE TEST, AND COMPRESSION TEST ON BRICKS VARYING IN DEPTH FROM 2 TO 4 INCHES

TABLE 1.—*Rattler test results on brick used to determine effect of size*

Sample No.	Percentage of loss in weight				
	Thickness of brick (inches)				
	2	2½	3	3½	4
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	22.6	17.9	18.6	17.4	17.3
2	22.0	21.4	18.3	16.8	16.5
3	23.1	18.3	19.7	16.8	16.4
4	21.9	18.3	19.7	16.5	16.3
5	22.8	19.1	18.7	16.2	17.0
6	22.8	19.3	19.7	17.0	17.1
7	22.7	19.0	18.6	17.2	16.3
8	22.9	17.6	19.3	17.0	17.2
9	24.7	19.2	19.1	18.0	17.4
10	23.3	17.9	18.2	17.4	16.6
Average	22.9	18.8	19.0	17.0	16.8
Maximum	24.7	21.4	19.7	18.0	17.4
Minimum	21.9	17.6	18.2	16.2	16.3

A very brief study of these tests is in order with a view to determining what differences in quality exist between the various sizes. It will be seen at once that the tests are not altogether consistent. For instance, the 4-inch size has a considerably higher crushing strength than any of the other sizes. This difference, however, is not reflected in any of the other tests, which is, of course, not surprising when we consider that each of these tests measures a specific property of the brick. When taken as a whole, however, the results give a general idea of the relative quality of the five sizes under consideration which, in this case, is what we are after. Assuming for the moment that the resistance of brick in the rattler is influenced by both hardness and toughness and that these qualities are measured individually probably better by the hardness test and the transverse test than by any of the others, it is found that from the standpoint of hardness the 2-inch brick are considerably softer than any of the other sizes; the 3-inch are next; and the 2½-inch are the hardest. In transverse strength the 2½-inch size ranks highest; the 3½-inch next; with the 2-inch, 3-inch and 4-inch practically identical. Taking both tests into consideration, the brick may be tentatively rated relatively as to quality about as follows: 2½-inch, 3½-inch, 4-inch, 3-inch, 2-inch. It should be borne in mind, of course, that these differences are not large numerically, and are of significance only because of the special use to which the rattler tests will now be put.

EFFECT OF SIZE ON RATTLER LOSS

In Figure 2 are plotted the average losses in pounds for each size of brick against the initial weight of the brick charge. There are also plotted two series of points, one above and the other below the actual losses, which show what the losses would have been if they had been (1) directly proportional to the number of linear inches of edge exposed to wear, and (2) directly proportional to weight (or volume). It will be seen that the actual curve is somewhat below a line bisecting the angle formed by the two theoretical curves, which indicates that the correction to be applied to the small sizes is somewhat smaller than has been commonly used on the assumption that the loss takes place principally on the edges and corners of the brick.

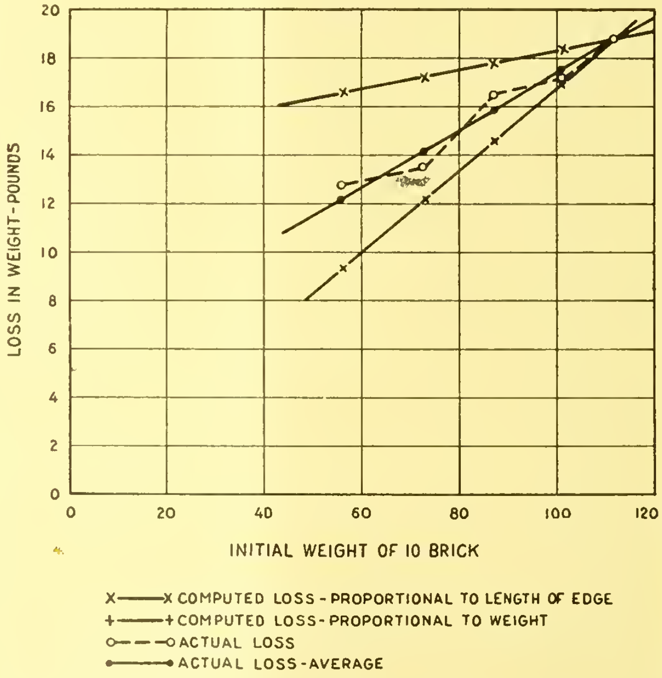

FIG. 2.—DIAGRAM SHOWING ACTUAL LOSS AND THE AVERAGE ACTUAL LOSS OF VARIOUS SIZES OF BRICK IN THE RATTLER TEST. THERE IS ALSO SHOWN WHAT THE LOSSES WOULD HAVE BEEN HAD THE LOSS BEEN PROPORTIONAL TO THE LINEAR INCHES OF EDGE EXPOSED TO WEAR AND TO THE WEIGHT OF BRICKS

Returning now to a discussion of the actual losses, it is found that the plotted points do not lie on a straight line as they would were the brick all of the same quality and the differences in loss due entirely to the effect of size. Plotting the average line, it is found that the losses for the 2½-inch and 3½-inch sizes lie below the line, whereas those for the 2-inch and 3-inch sizes lie above the line. This grouping is exactly what would be expected as a result of our study of the relative quality of the five sizes based on the hardness and transverse tests, and indicates that these tests are a reliable measure of those properties of the paving brick which are affected by the rattler test.

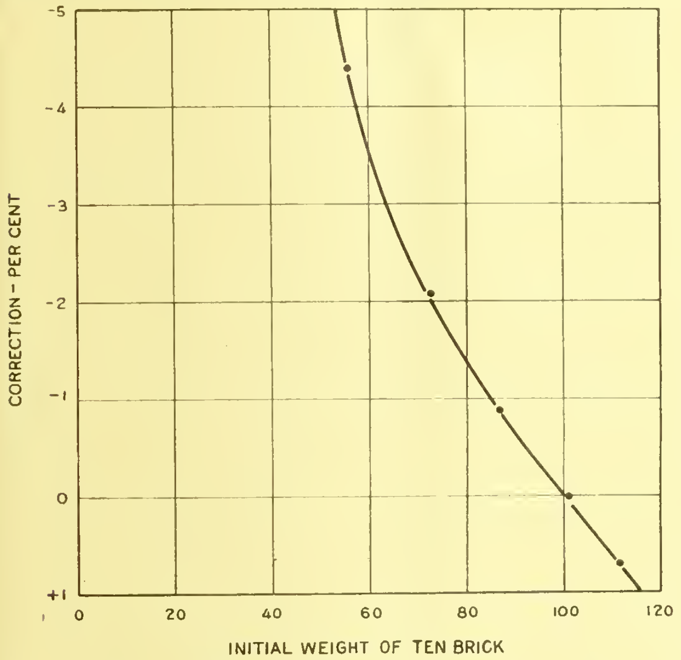

Fig. 3.—Diagram Showing Percentages to be Added to or Subtracted from Observed Rattler Test Loss to Give Results in Terms of "Standard" 10-Pound Brick

One would seem justified, therefore, in drawing the average line as indicated on the chart, and recomputing the various percentages of loss from the corrected losses obtained therefrom. This gives a series of values the same as would have been obtained experimentally had all the brick been of the same quality. Using these values, a correction curve has been plotted as shown in Figure 3. By means of the chart it is possible to determine what percentage shall be added to or subtracted from the observed rattler loss to give the equivalent value in terms of a "standard" 10-pound brick. The corrections, it will be observed, are based entirely on weight and not on nominal size. Although it might be more convenient to use the latter method, it would seem unwise to do so because of the wide variations in actual size of brick of the same nominal size. For instance, among eight brands tested by the bureau in connection with this work, it was found that for the 2½-inch size the actual weights of 10 brick varied from 64 to 75 pounds, whereas for the 3-inch size the corresponding variation was from 82 to 89 pounds. These differences, of course, are accounted for in part by differences in specific gravity of the material. In general, however, the differences in size appear to be more pronounced, as will be observed by noting Figure 4, in which are plotted the average weights and corresponding volumes for each of the brands and sizes of brick tested.

It will be observed that the points lie fairly well on a straight line, those above the line indicating the lighter brick, whereas those below the line indicate the heavier brick. Of course, a correction based on the volume of brick would be the most rational method because it would eliminate variations in the volume-weight relations owing to differences in specific gravity. However, the weight determination is much simpler and more readily made, and it would not appear that the small changes in specific gravity which normally occur in well-burned paving brick would seriously affect the accuracy of the corrections.

SUPPLEMENTARY CHECK TESTS

In order to determine to what exent the proposed correction curve could be applied in actual work, a series of check tests was run on a number of different brands of paving brick submitted by the manufacturers at the request of the National Paving Brick Manufacturers Association. Each brand was represented by at least two sizes, and in some cases by three sizes. Control tests for hardness and for modulus of rupture were likewise run, in order to check the quality of the brick by tests independent of the size factor. Unfortunately, the number of brick available for these supplementary tests was so limited that it was found impossible to obtain a sufficient number of tests for hardness and modulus of rupture to obtain representative averages.

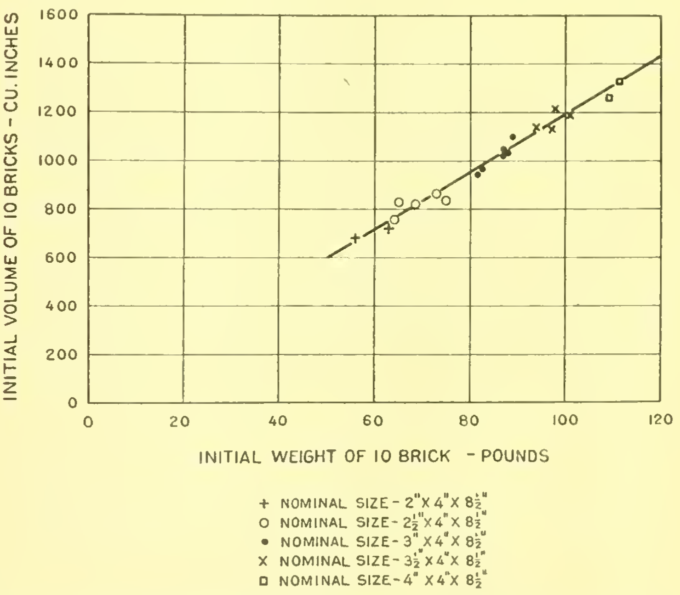

Fig. 4.—Diagram Showing Variation in Weight and Volume of Groups of Ten Brick of Various Sizes and Brands Prepared for the Rattler Test

The results of the rattler tests on six lots of brick, representing four brands of wire-cut-lug, shale brick and two brands of plain, wire-cut, fire-clay brick are shown in Table 2, together with their initial volumes and weights and the corrected percentage of loss derived from the use of the correction curve shown in Figure 3. Each value for per cent of wear represents the average of three tests. It will be observed that in

(Continued on page 107)

A DEVICE FOR MEASURING PRESSURE USED IN MOLDING CEMENT MORTAR BRIQUETTES

Reported by F. H. JACKSON, Engineer of Tests, and D. O. WOOLF, Junior Materials Engineer, U. S. Bureau of Public Roads

THE pressure exerted by the operator in molding the standard 1:3 cement mortar briquette probably has as great an influence on the tensile strength as any other single factor. Notwithstanding this fact, the present standard methods of testing Portland cement do not provide for the application of any definite pressure, the matter being governed by general reference only. This naturally has resulted in considerable variation in the technique employed by different operators and probably has accounted in part, at least, for the wide divergence in strength tests frequently reported when identical samples are tested in different laboratories.

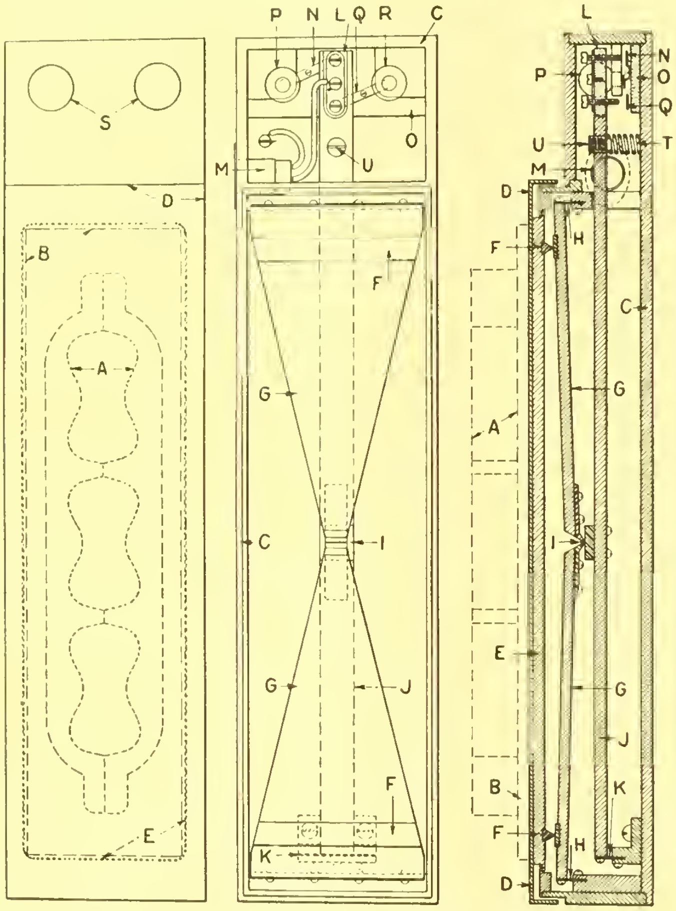

FIG. 1. DETAILED DRAWING OF PRESSURE REGISTERING APPARATUS

Recognizing the desirability of controlling this as well as many other details of procedure subject to personal interpretation, the American Society for Testing Materials prepared and published a year ago a "Manual of Cement Testing," in which the various methods of testing were described in much greater detail than in the specification proper. In that portion of the manual referring to the molding of briquettes, a pressure of 15 pounds applied to the surface of the briquette was suggested. This value was taken by the committee from the recently proposed master specification for Portland cement of the United States Government and was probably suggested originally by the United States Bureau of Standards. Practically the same pressure, modified so as to call for a minimum of 15 pounds and a maximum of 20 pounds, is provided for in the new tentative specification for Portland cement of the American Society for Testing Materials, adopted provisionally at the June, 1926, meeting. This specification, if adopted by letter ballot of the society, as it probably will be, will automatically become the new American Society for Testing Materials standard some time before the close of the present calendar year.

In the new specification, the requirements for tensile strength of briquettes have been raised from 200 pounds per square inch to 225 pounds per square inch at 7 days and from 300 pounds per square inch to 325 pounds per square inch at 28 days. This increase in the strength makes it all the more important that the technique of testing, in so far as those factors which influence the strength are concerned, be very rigidly controlled. The new values represent more nearly the true tensile strength of many normal Portland cements than the old, so that incorrect methods of testing, which tend to give low results, will undoubtedly cause more rejections. Then too, it is well recognized that many operators, especially those of long experience, have become accustomed to using a pressure considerably greater than that provided for in the new specification. Under the proposed specification, these operators will be required to use a pressure of from 15 to 20 pounds, which will undoubtedly result in lower strengths being reported, still further reducing the factor of safety.

PRESSURE MEASURING INSTRUMENT DESIGNED

Anticipating the need of a simple yet fairly accurate device for measuring the pressure exerted in molding briquettes, the Bureau of Public Roads has designed an instrument for this purpose.[1] The instrument consists essentially of a platform to support the briquette mold, resting upon a multiple lever system which operates against a small coil spring. Two contact points are so arranged on the lever arm that the degree of compression of the spring is indicated by the closing of one or both of two lamp circuits. The initial compression of the spring may be varied so that pressures of from about 10 to 40 pounds may be registered.

A detailed drawing of the instrument is shown in Figure 1, reference to the various parts being indicated by letter. In this drawing, *A* represents a three-gang briquette mold resting upon a glass plate, B, which in turn rests upon the platform of the instrument, *D*. The pressure exerted is transmitted through the knife edges and hardened bearings, *F*, mounted on the transmitting levers, *G*, which have steel ribbon supports, *H*, and steel knife edge points *I*, resting upon the operating lever, *J*. This lever has a steel ribbon support, *K*, at one end, and at the other end, a fiber insert, *L*, carrying three screws, the center one connected by wire to a standard electrical connection, *M*, of dry battery, the other wire being grounded to the case. When the minimum allowable pressure has been applied, one of the remaining screws makes contact with a spring, *N*, mounted on a fiber block, *O*, lighting a white electric lamp, *P*. When the maximum allowable pressure has been reached, the other screw comes in contact with a similar spring, *Q*, lighting a red electric lamp, *R*. Both lamps may be observed by the operator through the holes in the cover, *S*. A coiled spring, *T*, adjusted by the screw, *U*, returns the operating lever to its initial position when the pressure is released.

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[1] The original design of this apparatus was prepared by E. B. Smith, formerly engineer of tests, Bureau of Public Roads.

The instrument is calibrated by placing a static load, including the weight of the briquette mold, etc., upon the platform, equivalent to the minimum pressure it is desired to register. The adjusting screw over the spring connected to the white lamp is then turned until contact is just made. An additional load, making the total equivalent to the maximum allowed, is then placed upon the platform and the screw over the spring connected with the red lamp is similarly adjusted.

This device has been in operation in the laboratories of the Bureau of Public Roads for several months and has proved very satisfactory. It should be possible to regulate the applied pressure by means of this device to any specified value plus or minus two or three pounds. In other words, such tests as the bureau has made would indicate that, by the use of the instrument, it is possible to mold briquettes conforming to the new standard specification in which the minimum pressure is specified at 15 pounds and the maximum pressure at 20 pounds.

EFFECT OF VARYING PRESSURE ON THE STRENGTH OF BRIQUETTES

To determine the variation in strength resulting from molding briquettes at different applied pressures, a series of tests was recently conducted, using several different pressures. Two series of test specimens were made, with molding pressures and number of applications to each briquette face as follows:

Pressure, pounds	Number of applications per briquette face	Pressure, pounds	Number of applications per briquette face
15-20	12	15-20	18
20-25	12	20-25	18
25-30	12	25-30	18
30-35	12	30-35	18

Tests were made at the age of 7 and 28 days. The two series were then repeated with a second brand of cement and tests made at the age of 7 days. Each average is the result of 12 breaks. The average results of these tests are shown in Table 1.

Table 1.—*Effect of pressure and number of applications on average tensile strength of cement*

Applied pressure, pounds	Number of thumbings per briquette face	Tensile strength—pounds per square inch		
		Cement A		Cement B
		7 days	28 days	7 days
15-20	12	245	365	280
20-25	12	250	370	285
25-30	12	260	370	290
30-45	12	260	370	290
15-20	18	260	370	285
20-25	18	260	370	300
25-30	18	265	390	290
30-35	18	270	385	300

Considering the several lots of 12 specimens each, it will be observed that the average tensile strength increases in general with increase in the applied pressure and with the number of applications of pressure. This would, of course, be expected. It should be noted, however, that the increase in strength is not quite so large as has commonly been supposed. For instance, tests at 7 days, using 12 applications with a 15 to 20 pound pressure, gave in the case of the first cement an average of 245 pounds per square inch, whereas a pressure of 30 to 35 pounds gave an average of 260 pounds per square inch, or an increase of 15 pounds. The corresponding variation in strength at 28 days was only 5 pounds, the 15 to 20 pound pressure giving an average of 365 pounds and the 30 to 35 pound pressure an average of 370 pounds. Likewise, increasing the number of applications of pressure from 12 to 18 gave an increase in strength of approximately 10 pounds per square inch for both periods. Similar variations may be noted in the case of the second cement. In general it was found that increasing the pressure from about 15 to 30 pounds resulted in an increase of strength of from 10 to 15 pounds, the

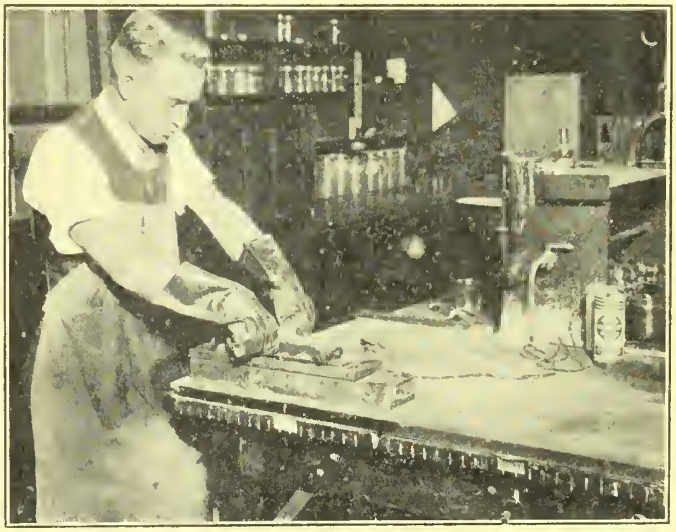

Fig. 2.—Molding Briquettes, Using the Apparatus for Control of Pressure

greater divergence being at 7 days, whereas an increase in the number of thumbings for each briquette face from 12 to 18, resulted in an average increase of strength of about the same amount.

In Table 2 are shown the mean as well as the maximum deviations of individual breaks from the average of 12. It will be noted that in so far as concordance of results is concerned, there is little to choose between the various pressures tried. The same comment applies to the tests made with 12 applications of pressure as compared with those made with 18 applications. It will be noted that in many cases the maximum deviation from the average exceeded the 15 per cent allowed by the specification in routine testing.

Detailed working drawings of the instrument described in this paper have been prepared and will be furnished upon request.

Table 2.—*Deviation of individual test results from the average values*

Applied pressure, pounds	Number of thumbings per briquette face	Deviation from average tensile strength					
		Cement A				Cement B	
		7 days		28 days		7 days	
		Mean	Maximum	Mean	Maximum	Mean	Maximum
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
15–20	12	6.8	14.3	8.7	16.4	4.4	8.9
20–25	12	4.7	10.0	5.4	17.8	4.8	12.3
25–30	12	5.2	15.4	6.0	14.9	7.1	12.1
30–35	12	5.4	9.6	6.0	16.2	4.5	8.6
15–20	18	4.5	13.5	5.9	13.5	5.5	14.0
20–25	18	4.8	15.4	4.9	13.5	3.9	6.7
25–30	18	5.8	13.2	4.4	11.5	4.4	6.9
30–35	18	5.1	14.8	6.2	19.5	3.8	8.3

THE STRENGTH OF MORTAR AND CONCRETE AS INFLUENCED BY THE GRADING OF THE SAND

Reported by J. G. ROSE, Materials Engineer, United States Bureau of Public Roads

IN CONNECTION with the testing of materials for Federal-aid highway projects in Colorado a study has been made of the relation between the grading of sand for use in concrete and the strength developed in mortar and concrete; and the study has led to the development of a graph which may be used as the basis for a preliminary judgment of the quality of sands proposed for use.

The samples of sand and gravel, or crushed rock, upon which the study was based were contributed by the State highway department of Colorado as materials to be tested for use on Federal-aid projects. Approximately 200 samples of sand and gravel are represented by the study. The source of the materials was widespread, almost every county in the State having contributed one or more samples.

The testing work was done by the Pierce Testing Laboratory, of Denver, and standard methods of testing concrete materials approved by the American Society for Testing Materials were followed in making all tests. Standard briquettes of 1:3 mortar were used for the tensile tests, and 6 by 12 inch cylinders of 1 : 2 : 4 mix were used for the compression-test specimens. The consistency of the concrete was such as to show a slump of from 1 to 2 inches as determined by the standard slump-cone method. All observations are based upon the 28-day strength of the specimens, both in tension and in compression.

While assembling the test data for the study it was observed that the strength of the 6 by 12 inch cylinders varied from about 1,500 pounds per square inch to a little over 3,500 pounds per square inch. In order to observe the variation in grading between the high and low strengths, the test reports were divided into four groups, each group having a range in strength of 500 pounds; and after computing the average grading of the maximum strength group (3,000 to 3,500 pounds per square inch) and plotting this average grading upon the graph, it was observed that part of the reports in the remaining three groups fell above the maximum strength curve, and part fell below it. The three groups were, therefore, divided again into two groups each, depending on whether the grading of the samples fell above or below the maximum strength curve. Samples falling partly above and partly below the maximum strength curve were listed in both groups; and the average gradings for the six groups thus obtained were then computed and plotted on the graph. The groups thus established, according to grading, were then averaged for tensile strength.

WHAT THE GRAPH SHOWS

The curves derived for the above averages as shown in Figure 1, lead to the following conclusions:

1. That there is an ideal grading of sands which will produce maximum strength in concrete.
2. That the ideal grading curve assumes an arched form showing a predominance of the material retained upon the coarser sieves.
3. That for a given mix, there is a practical limit to the quantity of material passing each size sieve, where a given strength of concrete is required.
4. That an exceptionally high tensile strength of sand in 1:3 mortar is not necessarily associated with a high compression strength of the same material when mixed with the average coarse aggregate in concrete, hence the tensile strength is not a proper gauge of the quality of a sand for concrete.

Justification of the relation between the grading and strength of sands as shown by the curves is dependent upon a combination of coordination of several well known factors or theories of concrete. The maximum strength curve is, doubtless, associated with maximum density, or minimum voids, in the combined aggregates. The decline in strength as the grading becomes finer is associated with increased surface area, and a corresponding increase of voids. As the grading of the sands becomes coarser than that shown by the maximum strength curve, the probability of increased strength indicated by the surface-area theory is overcome by a tendency of the coarser grains of sand to wedge themselves in between the coarse aggregate, thus increasing the voids to such an extent that a deficiency of mortar is produced. Hence, a decline in compression strength is recorded with the increase in coarseness of the sand. This, at least, is one explanation which comes to mind. Probably there are others. The relation of the grading curves to the tensile strength developed in 1:3 mortar bears out the surface-area theory; that is, that the finer the sand the greater will be the surface area to be covered by a unit volume of cement, hence, the weaker the bond.

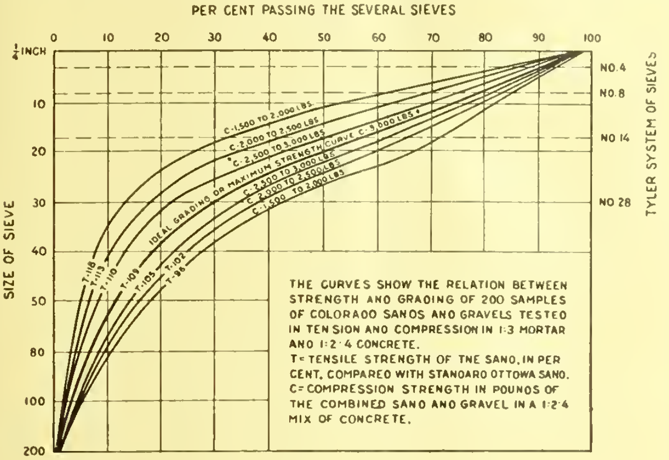

FIG. 1.—SAND ANALYSIS CHART

UTILITY OF THE GRAPH

By plotting the sieve analysis of a sand upon the graph, a ready means of visualizing the quality of the material for concrete is produced. The area between the upper and lower curves on the graph forms a practical safey zone for the grading of acceptable sands. If the plotted grading of a sand falls outside of this area, in whole or in part, there is but little chance that it will pass standard specification requirements without increasing the proportion of cement.

In making a materials survey for a project, selection of the best source of supply will be greatly facilitated by a comparison of the mechanical analyses of the samples when plotted on the graph. As a precaution in making selection of a sand, it should be realized that several other factors in addition to the grading affect the strength of sands. Variation in structure and soundness, and the presence of organic matter, silt, clay, acids, alkali, and other foreign substances all have their influence on the strength of the sand in concrete. Hence, a considerable variation in strength from the average strength curve for each group shown on the graph should be expected. Eliminating these factors the range of grading for any given strength and mix should be small. Final selection, of course, should always be determined by a more complete laboratory test. But once a complete laboratory determination for the quality of a sand has been made, and the mechanical analysis plotted, any change in quality due to variation in grading is easily detected by a screen analysis made in the field during the progress of construction and plotted on the graph.

(Continued from page 103)

TABLE 2.—*Check tests of paving brick*

Lot No.	Nominal size	Volume of 10 brick	Weight of 10 brick	Rattler loss	Corrected loss	Total variations within lots	
						Before correction	After correction
	Inches	*Cubic inches*	*Pounds*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	3	1,009	87	24.2	23.3		
	2	720	63	28.1	24.8	3.9	1.5
2	3½	1,219	98	19.9	19.8		
	3	1,113	89	21.3	20.6	1.4	.8
3	3½	1,128	97	16.1	15.9		
	3	1,028	87	16.7	15.8		
	2½	838	75	17.8	15.9	1.7	.1
4	4	1,266	109	17.0	17.5		
	3	974	83	17.9	16.7		
	2½	762	64	18.3	15.2	1.3	2.3
5	3½	1,141	94	17.1	16.8		
	2½	831	65	21.0	18.0	3.9	1.2
6	3½	1,156	94	19.7	19.3		
	2½	811	64	23.2	20.1	3.5	.8

all cases but one the differences in percentage between the corrected losses for the different sizes in any given lot are considerably less that the corresponding differences before correction. In only one case, however, that of lot 3, have these differences entirely disappeared, indicating that a certain amount of the original variation was due to differences in the quality of the brick. This is not surprising when it is remembered that no special effort in any case was made to obtain brick of exactly the same quality. In one case, that of lot 4, the results of the individual rattler tests were very erratic, indicating an extremely nonuniform product, which may account for the wide variations of the results obtained on this lot.

It is felt that the correction curve obtained as a result of this series of tests is sufficiently accurate for all practical purposes; and the writer has recommended that a table of correction factors based thereon be considered for use in connection with any revision of the present standard rattler test for paving brick. The following addition to paragraph 13 of the standard specifications for paving brick has been suggested:

> The result obtained in the original calculation shall be corrected by adding to or subtracting from it a factor depending upon the initial weight of the brick charge in pounds. This factor shall be obtained from the following table:
>
> | Original weight of 10 brick | Correction to be applied to percentage of wear obtained by test | Original weight of 10 brick | Correction to be applied to percentage of wear obtained by test |
> |---|---|---|---|
> | *Pounds* | *Per cent* | *Pounds* | *Per cent* |
> | 105 to 115 | 0.5+ | 75 to 79 | 2.0− |
> | 95 to 104 | .0 | 70 to 74 | 2.5− |
> | 90 to 94 | .5− | 65 to 69 | 3.0− |
> | 85 to 89 | 1.0− | 60 to 64 | 3.5− |
> | 80 to 84 | 1.5− | 55 to 59 | 4.0− |
>
> The final corrected value, together with the observed value and the correction factor, shall be reported.

MORE ACCURATE TESTS OF REINFORCING BARS

Reported by **D. O. WOOLF**, Junior Materials Engineer, U. S. Bureau of Public Roads

IN TESTING steel reinforcement bars, it has been the custom in many laboratories to assume that the cross-sectional area of the bar is that given in the manufacturer's tables. It has been found that, in the case of the deformed bar, the nominal area seldom agrees with the actual area, and that occasionally the difference is so great that serious errors may result from the assumption of equality. To insure more accurate tests of the physical properties of steel, the physical laboratory of the Bureau of Public Roads has adopted the practice of making planimeter measurement of the cross-sectional area of all concrete reinforcement bars submitted for test. The higher allowable unit stresses used in designing at the present time together with a desire for more rigid laboratory control have demanded that this greater care be used in determining the bar cross section.

The method used in the determination of the actual area is simple. The test specimen is cut through at right angles to its axis, and the cut surface is filed smooth. An impression of this surface is made by the use of carbon paper or stamp pad, and the area of the print is determined by polar planimeter measurement. In the case of bars of very irregular shape, several different sections are measured.

The standard specifications for billet-steel concrete reinforcement bars (serial A15–14) of the American Society for Testing Materials permit the testing of bars turned down to a uniform cross section. This method of testing deformed bars has the objectionable feature of requiring the use of a lathe which may not be available at the laboratory. By the use of the planimeter the cross-sectional area may be determined so accurately that the turning down to uniform diameter will not be necessary.

Table 1 shows certain data from some of the more recent tests made by the bureau. The unit tensile strength has been computed using both the nominal and actual cross-sectional areas. In several cases, had the nominal area been used the bar would have been classed as other than its true grade of steel. For example, samples 6 to 8, inclusive, were submitted as ½-inch round bars. Measurement of the cross-sectional area gave 0.25 square inch in each case, and computation of the test values gave unit tensile strengths corresponding to the structural grade. Using the nominal area of a ½-inch round bar, unit tensile strengths in the intermediate grade were obtained. As the specifications for this project required the structural grade, the steel would have been rejected had the nominal area been used.

TABLE 1.—*Comparison of actual and nominal cross-sectional areas of bars and unit strengths based on each*

Sample No.	Size and shape of deformed bar	Area of cross section		Breaking load	Unit tensile strength		Error in nominal unit tensile strength	
		Nominal	Actual		Nominal	Actual		
		Sq. ins.	*Sq. ins.*	*Pounds*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent*
1	½-inch square	0. 25	0. 20	12, 140	48, 560	60, 700	12, 140	20. 0
2	⅜-inch round	. 11	. 16	10, 990	99, 910	68, 690	31, 220	45. 4
3	½-inch round	. 20	. 25	15, 320	76, 600	61, 280	15, 320	25. 0
4	½-inch square	. 25	. 23	14, 590	58, 360	63, 430	5, 070	8. 0
5	¾-inch round	. 44	. 41	24, 740	56, 230	60, 340	4, 110	6. 8
6	½-inch round	. 20	. 25	15, 620	78, 100	62, 480	15, 620	25. 0
7	½-inch round	. 20	. 25	15, 650	78, 250	62, 600	15, 650	25. 0
8	½-inch round	. 20	. 25	15, 560	77, 800	62, 240	15, 560	25. 0
9	¾-inch round	. 44	. 43	36, 260	82, 410	84, 330	1, 920	2. 3
10	1-inch square	1. 00	. 98	61, 820	61, 820	63, 080	1, 260	2. 0

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

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331C. Standard Specifications for Corrugated Metal Pipe Culverts.

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REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D-3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JUNE 30, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2.863,197.86	611.8	$ 12,256.313.63	$ 5,862,787.23	686.5	$ 5,058.420.96	$ 2,374,991.75	207.3				$ 3,248.478.16	Alabama
Arizona	9.580,133.43	5.016,119.94	613.8	1,369.744.82	847,652.41	116.0	1.401.980.83	945.372.27	81.4	$ 82,189.48	$ 50.225.99	12.2	2.757.878.39	Arizona
Arkansas	13.310.190.08	5.380,181.73	1048.9	5.054,354.42	2,286.516.62	274.1	4.758.782.28	2.248.758.94	308.8	389,196.91	193.512.38	31.1	1,496.834.33	Arkansas
California	22,346.175.99	10,719.249.61	894.8	4.796,420.91	2,284,342.69	163.2	11.715,699.34	5.695,750.56	319.9	223.161.61	133.896.90	18.2	3,239.675.24	California
Colorado	11,876.703.94	6.067,814.34	651.2	2.029.200.70	1.059,473.84	93.8	4.456,058.36	2,192,681.79	214.4	726,547.90	326.138.34	21.8	2.679,703.69	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	855,927.90	281,217.14	15.5	3,269,571.40	921,363.03	48.0	924,659.87	221,419.88	9.9	1.090,312.29	Connecticut
Delaware	4.281.559.81	1,496.190.65	107.1	636.492.48	285,474.95	17.2	1,115.536.81	470.577.90	28.2	623.535.35	218.310.90	15.7	3.503.60	Delaware
Florida	2,959,273.72	1,405.487.97	96.3	873,406.54	418,874.35	36.6	9.925,038.28	4,645.765.56	279.7				1.614,826.12	Florida
Georgia	20,156,002.37	9,406,366.46	1478.3	4,635,204.60	2,257,871.40	315.7	12,261,322.41	6,036,376.25	656.9	1,438,858.45	690,662.84	28.3	40.676.05	Georgia
Idaho	9,394.676.80	4.815.332.26	600.1	1.666.521.34	1,066,780.44	124.6	2.958.753.24	1.789.391.33	162.5	766,646.86	464.085.19	72.5	434.037.78	Idaho
Illinois	40,010.481.10	18,640,076.28	1236.2	4,106.130.76	1,979.919.46	141.5	6.290,805.18	3.030.099.67	213.5	683.153.78	331,828.09	24.1	5.850,274.50	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	3,310,253.22	1,609,669.51	112.2	15,966,296.59	7,437,580.19	428.0	1,532.011.46	763,564.44	51.7	1.831,085.18	Indiana
Iowa	27,272.285.21	11,107,492.99	1996.9	1.790,090.19	816.809.11	117.9	9,830,045.68	4.770,528.91	648.7	2.865.214.46	1,329,250.33	146.8	1,459.481.66	Iowa
Kansas	26.399,695.77	9,755.273.32	831.4	6.426,905.87	2.835,215.93	329.2	11.703,240.93	4.616.421.11	651.9	1,867.736.46	676,229.78	129.9	1.581,270.86	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	5,905,381.82	2,286,087.66	173.4	5,631,422.97	2,666,402.84	280.0	95,483.33	47,741.66	3.6	2,006,582.25	Kentucky
Louisiana	11.939.424.97	5,279.870.86	927.6	1.891,167.71	864.869.13	128.3	3.435.673.08	1.667.428.72	161.2	875.858.92	383.621.41	29.8	1.076.617.88	Louisiana
Maine	8.174.281.31	3.907.870.33	281.4	573.271.45	284.637.06	22.8	2,364.417.18	854.849.24	67.3	1.344.693.66	555.945.87	45.6	861,525.50	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,792,436.20	1,263,608.07	128.9	613,398.28	252,643.43	30.7	1,280,407.48	535,388.21	57.7	24,034.14	Maryland
Massachusetts	14.047,656.22	5,467,661.28	300.6	4.306.101.49	1,189,599.34	73.9	3,548.227.96	996.569.84	46.5	1.389.802.47	337,508.17	21.2	2.117.387.37	Massachusetts
Michigan	16.234.000.80	7,328.316.91	612.6	9.763.239.98	4,498,735.39	350.4	10,129.758.58	4.599.288.88	246.5	208.854.00	99,254.00	11.6	3,816.769.82	Michigan
Minnesota	30,415,685.89	12,738,642.04	2721.2	6,755,300.06	2,847,474.52	460.7	9,330,294.00	3,493,000.00	591.3	443.985.90	31,000.00	11.1	481.663.44	Minnesota
Mississippi	10.292,285.79	4.988,702.73	803.4	4,853.802.73	2,425.831.37	325.6	6,629.582.39	3,283,796.56	360.6	1.916,418.91	867,077.13	95.1	562.610.21	Mississippi
Missouri	17.368.156.57	8,219.411.43	1116.9	11,621.010.35	5,516,603.42	424.3	20.568.938.77	8.128.296.60	563.6	1,215.244.48	516.163.45	23.1	405.961,10	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	1,244,383.40	1,015,942.74	133.3	1,749,936.25	1,194,856.27	132.7	1,770,351.41	1,345,318.41	145.7	4,551,144.43	Montana
Nebraska	9.306.374.36	4,389.523.50	1570.6	2.227.027.26	1,084.679.02	197.7	13.349.102.13	6,510.012.74	1352.1	943,438.61	470,648.61	94.0	2,180,371.13	Nebraska
Nevada	4,917,465.69	3.088.299.78	357.3	2,640.729.82	2,042,634.81	181.5	3,250,322.24	2,798.063.96	346.1	254.858.63	222.534.71	48.4	643.681.74	Nevada
New Hampshire	4,165,687.86	1,986.226.87	208.1	826.870.74	391,223.20	29.5	866,830.33	396,691.78	26.0	232.305.40	104,291.27	6.9	291,058.88	New Hampshire
New Jersey	11,961.357.45	3.820,679.99	219.1	4,384.943.56	1,277,662.22	71.8	6,814,161.67	2.519,262.44	31.6	567,180.59	150,315.00	10.0	699.500.35	New Jersey
New Mexico	8,717.999.18	4.914,070.61	1081.3	3,686,338.59	2,425.586.77	345.7	1.349,543.87	873.541.00	100.1	567,172.13	457.488.46	34.0	2.301,699.17	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	14,626,510.12	5,682,880.66	365.5	32,340,520.00	8,967.680.20	567.4	9,943,103.00	2,201,500.00	136.6	4,964,057.61	New York
North Carolina	21.014.450.41	8.746.454.59	1119.8	5,994,969.06	2,430,883.35	138.1	8.035,569.05	3.484.782.13	190.2	635,619.50	317.809.75	26.1	737.276.18	North Carolina
North Dakota	10,829.263.82	5.268.930.47	1917.5	1,484.047.58	762.929.31	275.6	5.408.990.29	2.744.108.72	750.9	2,954.245.24	1.468.329.42	388.8	504.361.08	North Dakota
Ohio	41,572,252.81	15,244,993.93	1191.1	6,127,280.09	2,126,793.10	173.0	11,554,505.96	4,475,373.81	353.6	2,980,871.01	1,146,320.00	85.8	2,738,315.16	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	7.460.925.39	3,487,108.81	326.7	2.893,119.44	1.354.980.82	97.4	726.614.30	343,264.78	73.1	1,201,542.25	Oklahoma
Oregon	14,388.188.70	7,142,364.63	794.6	2,639.689.78	1,450,850.16	144.6	3.262,909.64	1.794.020.71	126.3	409,330.64	237.616.61	22.0	254,494.89	Oregon
Pennsylvania	43,054,836.19	16,222,023.97	850.3	18,311,315.61	5,338.708.07	338.5	25,913,665.91	7,325,756.54	529.7	2,405.915.24	753,286.81	57.4	1.699.006.61	Pennsylvania
Rhode Island	2,628.496.20	1,119,688.09	64.8	1.360.119.89	439,140.97	21.9	1.531.802.80	427,155.00	28.5	485,392.16	124.365.00	8.3	557,219.94	Rhode Island
South Carolina	11,163.347.84	5.121,267.54	1235.9	3.857.292.06	1,645,055.39	246.0	6,332.252.88	2.863.133.47	240.8	362.029.99	114,921.58	19.3	37.146.02	South Carolina
South Dakota	12,091.434.67	5.989,879.00	1447.9	5.376.938.52	2,613,947.97	733.3	3,716,153.62	1,864,529.03	633.3	551,764.94	299.132.42	119.8	399,301.58	South Dakota
Tennessee	13,789.140.98	6.732,079.77	497.9	7.835.490.59	3,544,504.25	282.1	7.630.941.58	3.490.104.05	248.4	1,513,770.60	628.578.00	24.6	885.324.93	Tennessee
Texas	54.120.970.83	21,057.940.12	3907.1	15.062,702.65	6,382.314.60	1013.1	19.070.379.25	8.428.039.53	907.4	2,418,044.93	1,101.193.14	61.4	3.636.943.61	Texas
Utah	6.259.159.41	3.818,836.91	423.1	1.994.018.62	1,279,603.77	123.3	1,688.956.94	1,269.611.66	154.0	785.929.62	599.447.91	49.4	851,278.75	Utah
Vermont	3.015,174.51	1,452,834.45	107.8	1.226.858.13	564.805.06	26.7	1,837,974.09	723.688.66	34.6	71,000.71	21,322.97	1.5	505,795.86	Vermont
Virginia	13,099.720.01	6,271.998.20	676.2	8.890.529.43	4,113,729.91	329.3	6.468.112.20	2.824.006.82	185.7	401.916.32	189,322.63	10.9	102,456.44	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,726,007.45	1,665,697.59	141.9	3,309,677.73	1,653,600.00	41.1	1,171,769.67	414,000.00	45.3	295,266.64	Washington
West Virginia	7,343,200.86	3.230,293.33	326.7	2.130.515.58	910.769.32	66.2	5.291.716.29	2.060.920.93	137.1	1,195,906.96	519,235.61	40.3	631,291.91	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1451.7	3.049.367.28	1,473,065.11	140.4	6.916.454.17	3.369.475.19	313.4	1.661,466.49	825.731.00	102.8	2.860,903.08	Wisconsin
Wyoming	8,809.819.33	4.739,096.67	982.0	2.118.483.23	1.301,790.38	151.5	2,635,904.29	1,663,408.02	213.7	126,205.97	81,023.00	10.4	780,955.93	Wyoming
Hawaii							1.050.897.93	312.635.18	15.9				787,517.82	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	226,552,043.54	100,524,357.58	10628.3	347,113,666.05	148,527,474.03	14355.1	56,029,885.69	22,839,821.04	2483.8	73,769,001.35	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 84.710.263.69 Federal aid $ 37,375,350.32 Miles 3393.8

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 6 ▽ AUGUST, 1926

THE NORWALK-DANBURY ROAD, CONNECTICUT

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A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 7, NO. 6 AUGUST, 1926 H. S. FAIRBANK, Editor

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Owing to the necessarily limited edition of this publication it will be impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, periodicals upon an exchange basis, and Members of both Houses of Congress. Others desiring to obtain "Public Roads" can do so by sending 10 cents for a single number or $1 per year to the Superintendent of Documents, Government Printing Office, Washington, D. C.

THE CONNECTICUT TRANSPORTATION SURVEY

DIGEST OF THE REPORT OF A SURVEY OF TRANSPORTATION ON THE STATE HIGHWAY SYSTEM OF CONNECTICUT[1]

CONNECTICUT was one of the first States to adopt the principle of State aid in highway construction. Its highway department was created in 1895 for the purpose of administering the State-aid appropriations. It was also one of the first to establish a definite system of connected trunk-line highways for construction and maintenance under the complete control of the State agency. This was in 1913 and with subsequent additions the system in 1923 embraced 1,566 miles, of which 1,114 miles were improved. By successive amendments of the law the State department has also exercised an increasing degree of control over the State-aid roads, and at present there is little to distinguish the State-aid roads from the trunk-line highways with respect to the State's participation in their improvement except that a portion of the cost of constructing the former is paid by local government units.

By 1923, when this study was made, the length of improved roads in the State highway system had grown to 1,780 miles and the amount of improvement better than macadam had increased to more than 480 miles. This improvement had been made with always inadequate revenues in the face of a constantly increasing traffic; the motor vehicle registration having increased from 23,200 in 1913 to 218,489 in 1924, an increase of 842 per cent. Its experience in this respect has been much the same as that of other States; but, because of its small area and its location between New York, Massachusetts, and Rhode Island, it has had to provide, on its main roads, highway service for a large traffic from adjacent States.

Although the State has been considered among the leaders in highway progress it was found that it was by no means keeping pace with the increase in traffic due principally to financial limitations. Old roads have been widened and strengthened, maintained to the limit of their serviceable life, and replaced as rapidly as possible with more adequate surfaces. In many cases the effort to prolong the life of the older roads beyond their economic life has resulted in heavy expense, yet limited funds permitted no other course. The same limitation is responsible for the fact that there remain on some of the most important roads sections of considerable length in the aggregate which are entirely inadequate for the present traffic.

Prior to 1923 it became evident that a large mileage of old road must be reconstructed, much of it involving widening of the road surface and improvement of the alignment which was designed to meet the needs of traffic of an earlier period. In addition to this a considerable mileage of new improvement was already needed and it was realized that still more would be needed to accommodate the rapidly increasing traffic. It was for the purpose of ascertaining the facts necessary as a basis for the framing of a program to meet these needs that the Connecticut highway transportation survey was undertaken jointly by the Bureau of Public Roads and the State highway department.

PURPOSE AND METHODS OF THE SURVEY

The general purpose of the survey was to obtain the traffic information necessary for the establishment of a definite plan of highway improvement based on the present and expected future traffic.

A classification of all highways on the basis of their relative traffic importance was felt to be the first need. Such a classification is required to determine the order in which the highways should be improved and the distribution of construction and maintenance funds over the highway system.

The selection of the most economical type for each highway is the next step. Such a selection must be based not only upon the present and expected future traffic density but also upon the type of the traffic units. The more important considerations are: (1) The present and estimated future density of traffic; (2) the ratio of the number of trucks to the number of all vehicles; (3) the relative number of trucks of large, medium, and small capacities; and (4) the maximum wheel loads and the frequency of heavy gross loads and wheel loads.

The final selection of the type of surface depends also upon physical considerations such as topography, drainage, soil and subgrade conditions, availability and cost of materials, as well as upon traffic considerations.

In addition to these purposes it was also planned to determine the place of highway transportation in the transportation system as a whole, and to establish the principles which should form the basis for the coordination of highway transportation with other forms in order to develop as a whole the most efficient transportation system.

The survey was begun in September, 1922, and continued for one year, during which time traffic data were recorded at the 57 survey stations shown in Figure 1. These stations, carefully located and designated before the beginning of the survey, were operated on an average of once a month by "recording" parties which recorded all passenger-car information and the principal types of motor-truck information, with the exception of the weights of the vehicles. Nine hours constituted the length of each operation at a station; the actual hours, however, varied with each operation and ranged between 6 a. m. and 9 p. m. A sufficient number of night operations were made to enable the correction of all traffic counts to an average 24-hour day.

In addition to the operation by recording parties, 8 of the 57 stations were operated by a "weight" party, which recorded all motor-truck traffic information, including weights of vehicles. These eight stations were located at key points on the principal highways in different sections in order that practices in motor-truck loading in various parts of the State could be determined.

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[1] The complete report of the survey undertaken jointly by the Bureau of Public Roads and the Connecticut State Highway Department has recently been issued. Copies may be obtained from the Bureau of Public Roads upon request as long as the free supply lasts or they may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C., at 30 cents per copy.

The average daily distribution of traffic on the important highways of Connecticut as developed by the survey is shown in Figure 2. It is evident that the routes connecting important centers of population and industry carry the heaviest traffic. The Boston Post Road from Greenwich to New Haven is the most important highway in the State and the same route from New Haven north to the Massachusetts line is almost of equal importance. In general, the traffic tends to vary directly with the population and industry served by the route and inversely as the distance from centers of population and industry.

With the exception of the route from Greenwich to Westerly, the general direction of the important traffic routes is north and south. Other east and west routes although of considerable importance are secondary to the north and south routes. The distribution of average daily truck traffic shown in Figure 3 is, in general, similar to the distribution of total traffic.

The width of the white space between the black lines of Figures 2 and 3 represents the average daily traffic throughout the year. But traffic varies greatly with the seasons of the year and the days of the week. Hence there is also shown in these figures by the width from outside to outside of the bordering black lines the maximum traffic which occurs at all points at certain periods. Considering all types of vehicles, Sunday is the day and August the month of maximum total traffic. The distribution of the total traffic on a Sunday in August is therefore presented in Figure 2 as an indication of the maximum traffic density. The density of the traffic on this particular day was 254 per cent of the average.

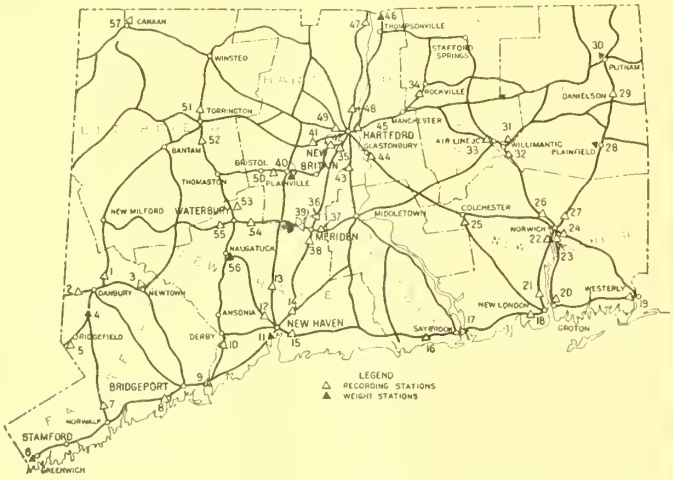

Fig 1.—Map Showing Location of Connecticut Traffic Survey Stations

Variation in truck traffic is considerably less than the variation in passenger-car traffic. The monthly variation in passenger-car traffic ranges from 29 to 193 per cent of the average. For truck traffic the corresponding range is from 68 to 122 per cent. Truck traffic is very uniform from Monday through Friday. On Saturday it is somewhat lower than on other week days, and on Sunday it is very low compared with other days of the week.

Maximum daily truck traffic, as shown in Figure 3, which represents a Friday in October, is approximately 146 per cent of the average daily truck traffic for the year. These figures represent the present normal peak load of traffic for which highway facilities are required. Unusual density of traffic on holidays or special movements on certain routes due to fairs, football games, and similar events will exceed in density the movements indicated on these charts. It is not necessary to consider these special movements in planning the highway program, but due allowance must be made for the expected increase in traffic during a reasonable period in the future.

In connection with the survey a map showing the distribution of population was prepared for comparison with the map showing the distribution of traffic. It is evident from this map that there is a relation between them. Of the 17 towns (the town in Connecticut corresponding roughly to the township in other States) having a population in 1920 of 640 or more per square mile, 16 are traversed by the seven routes which have been classified as the principal routes.

The areas adjacent to the principal routes are increasing in population at a more rapid rate than other sections of the State, the rate of increase varying from 17.2 to 39 per cent, whereas the rate of increase for the remainder of the State is only 4.7 per cent. The more rapid rate of population increase in the areas adjacent to the principal highways indicates the urgent need for the planning and construction of highways to serve future traffic needs in these areas.

The present distribution of population and the trends of population growth as reflected in the more rapid increase in the densely populated areas and the more rapid increase in urban than in rural population indicate that the present main traffic routes will continue to be the important routes and may increase in relative importance. The changes in relative importance of main traffic routes, as compared with those of secondary importance, must of necessity be slow and will not be important during the next few years.

The present trends indicate that minor traffic routes will with some exceptions continue to be of minor importance. The present areas of low population density and of decreasing or slowly increasing population will not become important traffic areas during the next decade.

MOTOR TRUCK CAPACITIES AND LOADING

The design of highways and the types of pavements to be selected are dependent not only upon the amount of traffic using them but also upon the type and weight of the traffic. The number of vehicles indicates the general importance of one route as compared with another and is the most important factor in considering highway width, parallel routes, and elimination of grade crossings and "bottle necks." But the determination of the amount of the traffic does not form sufficient or conclusive evidence for the final selection of highway design and type of pavement.

Analysis of traffic on the Connecticut roads reveals entirely different types of traffic on various highways. The first difference noted is a higher proportion of motor-truck traffic on one highway than another. This can be illustrated in the case of two highways, one of which connects two large industrial centers and the other a city and a pleasure resort. The latter highway would naturally be mainly a passenger-car route and be used less by motor-truck traffic (in proportion to total traffic) than the route connecting industrial

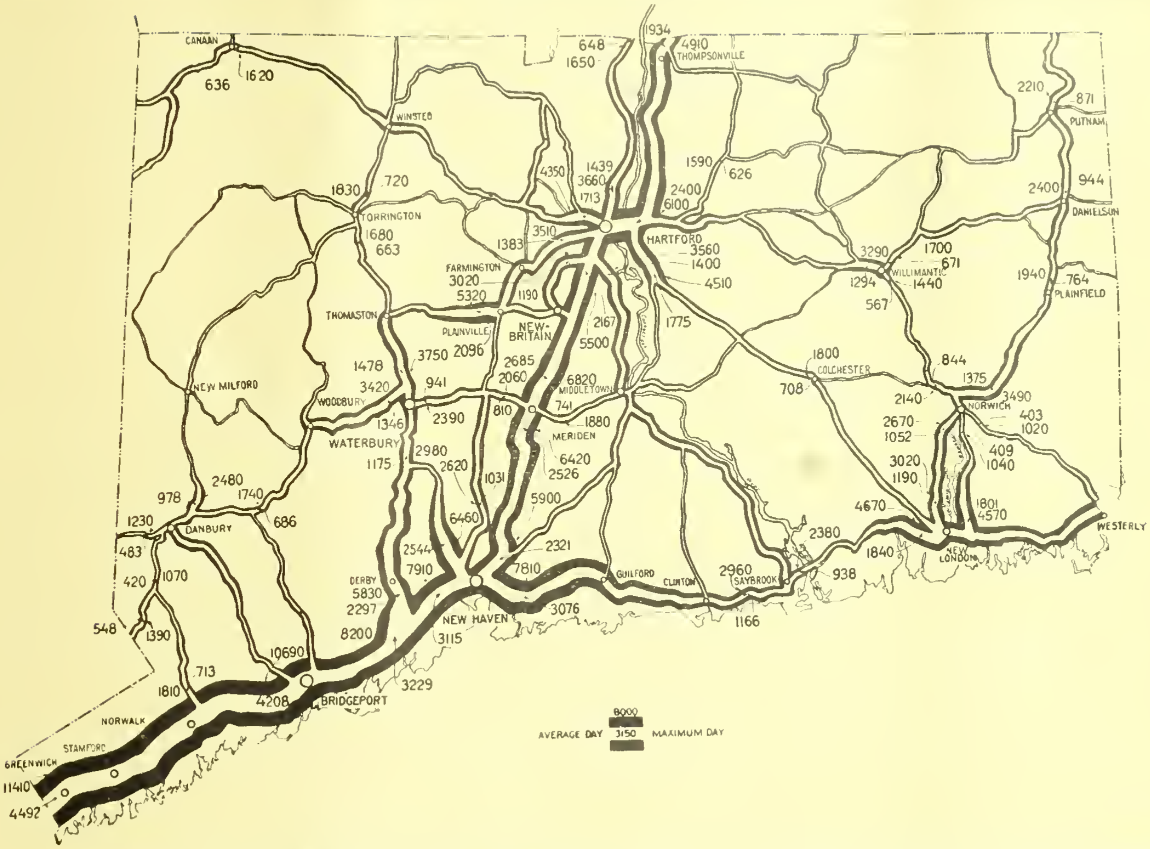

FIG. 2.—AVERAGE DAILY AND MAXIMUM MOTOR-VEHICLE TRAFFIC ON THE IMPORTANT HIGHWAYS OF CONNECTICUT

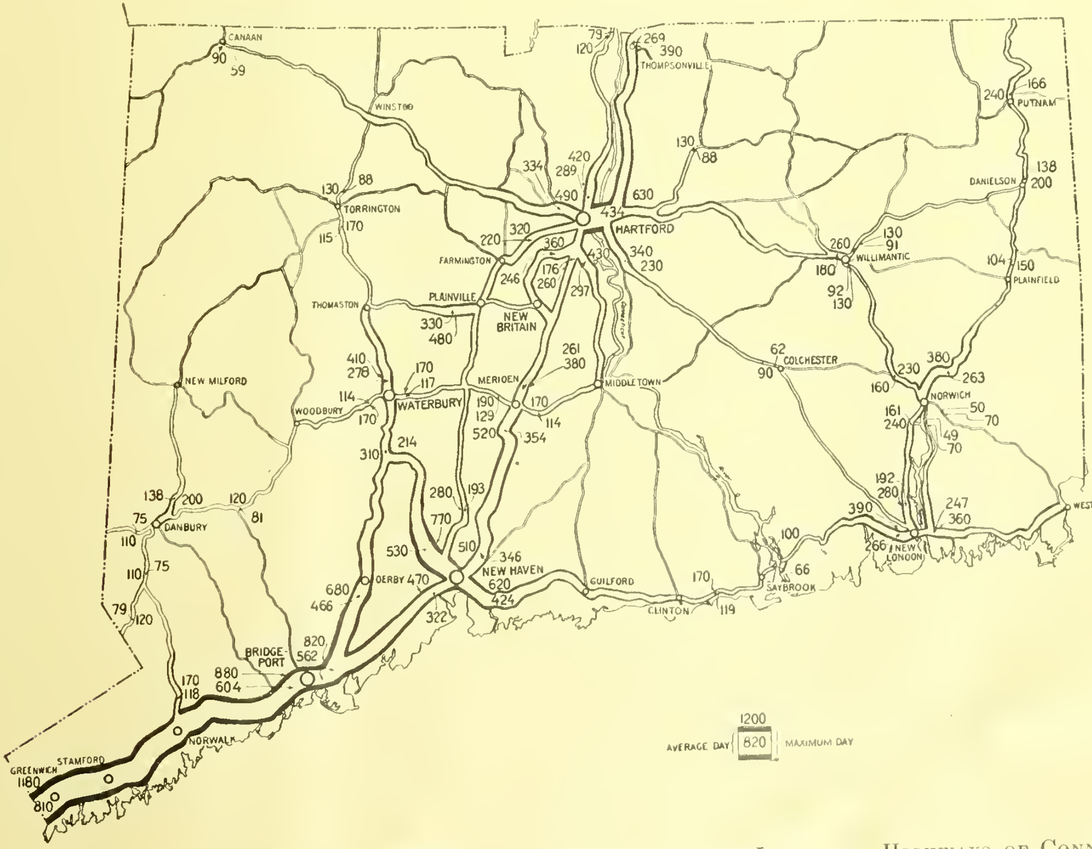

FIG. 3.—AVERAGE DAILY AND MAXIMUM MOTOR-TRUCK TRAFFIC ON THE IMPORTANT HIGHWAYS OF CONNECTICUT

centers. A more searching analysis reveals variation in motor-truck traffic itself; a preponderance of large-capacity trucks and heavy gross loads on one route and of small-capacity trucks and light gross loads on another.

Variation in the capacity, loading, and tire equipment of the vehicles causes corresponding variations in the effect upon the highways. Light, pneumatic-tired vehicles have less effect then heavy, solid-tired vehicles. An investigation by the Bureau of Public Roads into the effect of motor-truck impact upon highways, shows that a badly worn solid tire can deliver an impact seven times as great as the static wheel load. Pneumatic tires, on the other hand, seem to definitely limit the impact. In no case, using obstructions as high as 4 inches, has it been possible to record pressures under pneumatic tires greater than double the static weight.[2] In the choice of the design and type of pavement, therefore, these variations in type of traffic must be considered.

Variations in the rated capacities of motor trucks using highway routes are an excellent basis for determining the type of motor-truck traffic. Small-capacity trucks are designed to carry light loads and although overloading (loading beyond the rated capacity of the truck) is not uncommon, it can not be practiced beyond a certain degree. The rated capacity of a truck is found to bear a close relation to the average load transported by it; and the capacities of all trucks using a highway will bear a similar relation to the tonnage transported over the highway. The proportion of light, medium, and heavy trucks passing over the highways of a State is the primary factor in the determination of types of motor-truck traffic.

Additional evidence concerning motor-truck traffic is obtained by an analysis of net and gross loads and of rear-axle and wheel loads. This evidence is not only valuable in the selection of highway design and type of pavement but also in traffic control and the regulation of overloading.

THE HALF-TON TRUCK PREDOMINATES

One-half ton is the predominant capacity of motor trucks using the Connecticut highways, approximately one-fourth of all trucks recorded during the survey being of that size. Most important, however, from the standpoint of the weight and number of the vehicles, is the 5-ton capacity, which was represented in the traffic by one-tenth of the total number of trucks. Table 1 shows relatively the number of motor trucks of all capacities observed in the State as a whole during the survey period. The trucks of 1-ton capacity and under represent on the highways a much smaller percentage of the total traffic than they do of the total registration, while all other capacity groups are present in greater proportion in the traffic than in the registration lists. It is apparent, however, from Table 1 that, in the State as a whole, the motor-truck movement consists very largely of small-capacity trucks, although the heavy-truck movement is much heavier than the registration of such trucks would indicate.

TABLE 1.—*Distribution of observed motor trucks by capacity*[1]

Capacity of trucks	Percentage of total number of observed trucks	Capacity of trucks	Percentage of total number of observed trucks	Capacity of trucks	Percentage of total number of observed trucks
Tons	*Per cent*	*Tons*	*Per cent*	*Tons*	*Per cent*
½	25.1	2	9.7	5	10.2
¾	3.8	2½	3.7	5½	.4
1	18.4	3	1.7	6	.3
1¼	12.5	3½	6.7	6½	.6
1½	6.0	4	.7	7½	.2

[1] Based on 82,738 trucks.

It is possible from a study of the capacity of trucks using the highways of the State to classify certain routes, for purposes of highway design and surface type selection, as terminal highways and others as class A, B, and C highways. A terminal highway may be defined as a highway connecting large industrial centers, not widely separated between which there is a daily motor-truck traffic which consists of a large number of trucks of all capacities but especially of large-capacity trucks. Class A highways may be defined as carrying a smaller number of motor trucks of all capacities than terminal highways; class B highways as those the traffic of which consists mainly of small-capacity trucks; and class C highways as those over which the traffic of motor trucks of all capacities is relatively unimportant. In addition to the classification of highways, it is possible from a study of the movement of trucks of various capacities to establish certain general principles regarding the movement.

Figures 4 and 5 illustrate the movement of small and large-capacity trucks over the Connecticut highway system. Several significant features are observable in the movement illustrated by these figures. One of these is the great volume of ½ to 2½-ton trucks (fig. 4) observed around large centers of population and between adjacent large centers of population. This condition is apparent around Hartford, New Haven, and Bridgeport and is accounted for, in large part, by the fact that large cities and towns are distribution centers for commodities and because, in this distribution of commodities, the small-capacity truck is in general the most economical unit. The movement of small-capacity trucks is predominantly a local or short-haul movement. This is particularly true of the ½ to 1½-ton trucks, the average trip mileage of which ranges from 12 to 18 miles. Table 2 shows the average trip mileage of motor trucks of various capacities.

TABLE 2.—*Average trip mileage of motor trucks of various capacities*

Capacity	Average trip	Capacity	Average trip
	Miles		*Miles*
½ ton	12	4 tons	30
¾ ton	17	5 tons	45
1 ton	15	5½ tons	54
1¼ tons	18	6 tons	36
1½ tons	17	6½ tons	42
2 tons	27	7½ tons	60
2½ tons	34		
3 tons	29	All capacities	22
3½ tons	33		

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[2] Status of the Motor Truck Impact Tests of the Bureau of Public Roads, by C. A. Hogentogler, PUBLIC ROADS, vol. 5, No. 9, p. 14.

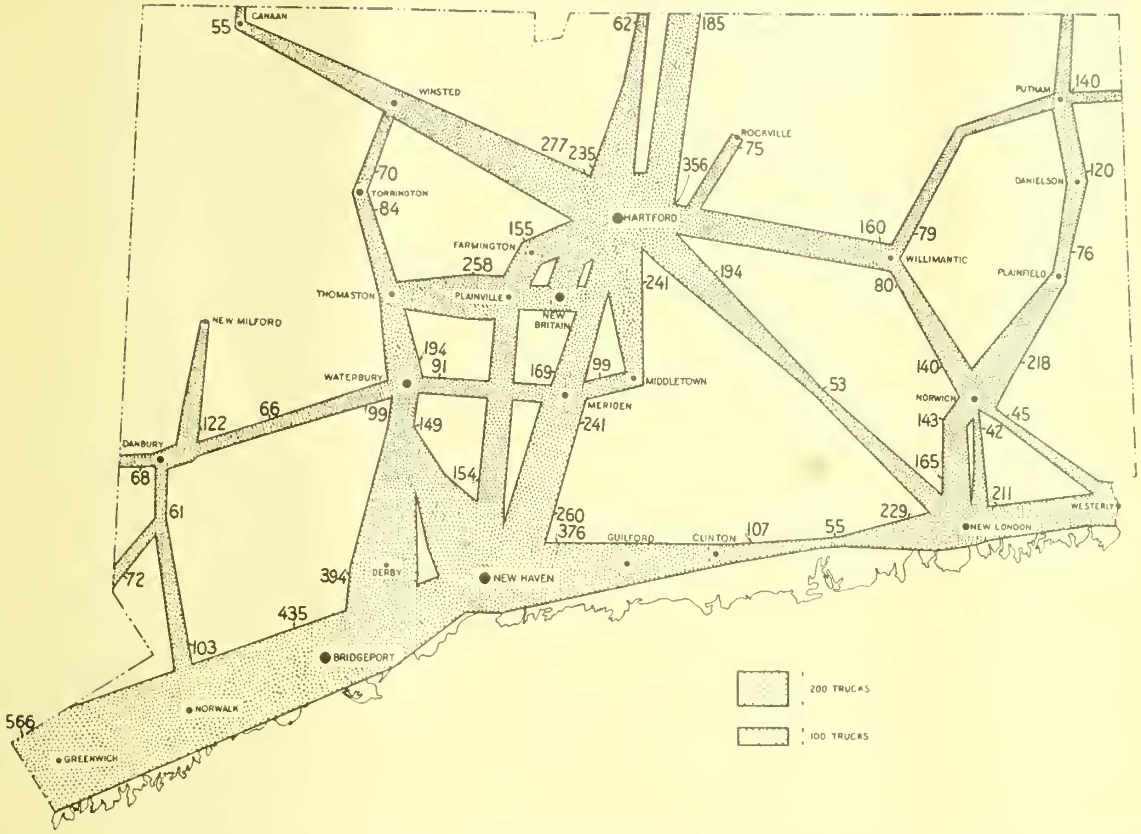

Fig. 4.—Average Daily Traffic of Small-Capacity Trucks (One-half to 2½ Tons) on the Important Highways of Connecticut

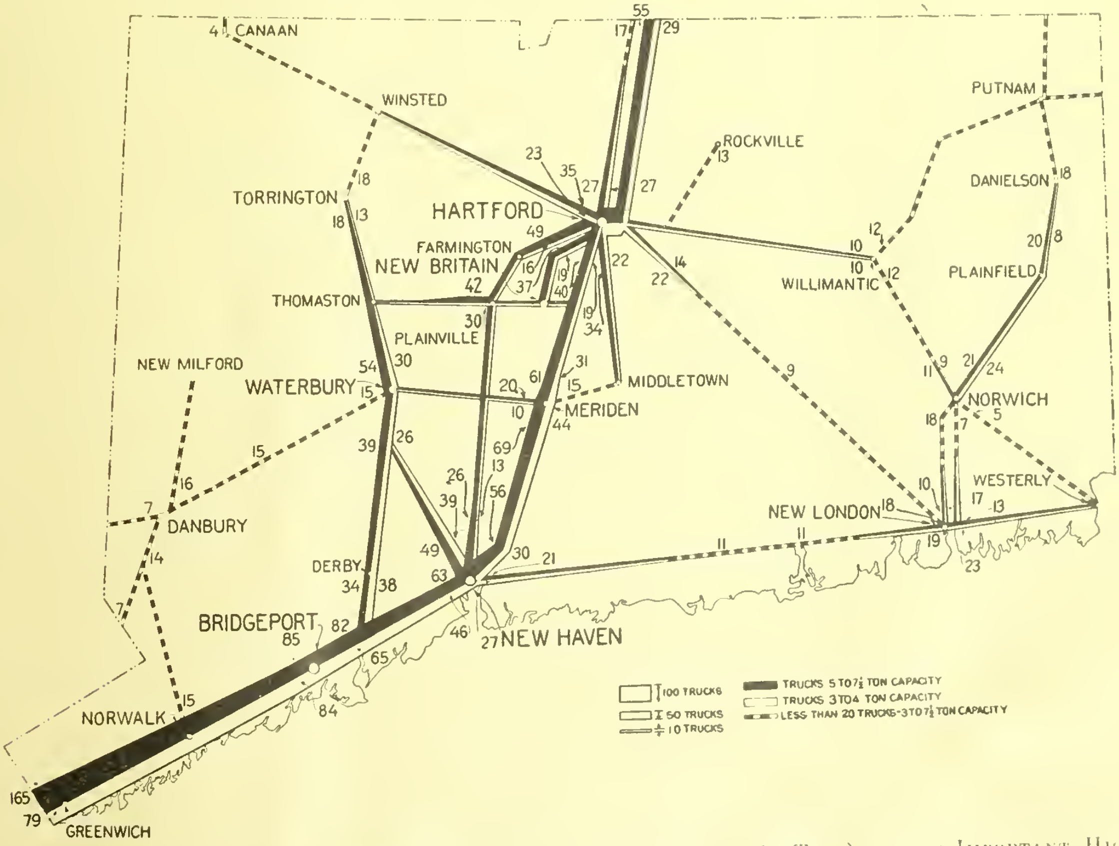

Fig. 5.—Average Daily Traffic of Large-Capacity Motor-Trucks (3 to 7½ Tons) on the Important Highways of Connecticut

The location of the State is a contributory factor to the long-haul movement of large-capacity trucks. Connecticut is a highly developed industrial area and its highways are the main traffic arteries between New York City and New England points, resulting in a large proportion of long-distance haulage by large-capacity vehicles.

TRUCK OVERLOADING PRACTICES

Approximately two-thirds of all motor trucks observed on the Connecticut highways during the course of the survey were loaded.

Comparing the large and small-capacity trucks, it was found that a somewhat larger proportion of the former were loaded. Of the total number of ½-ton trucks, for instance, 60.5 per cent were found to be loaded; while the loaded 5-ton trucks were 66.3 per cent of the total number. Although the differences between the proportion of loaded small-capacity and large-capacity trucks are not very great, it is apparent that the large-capacity trucks more frequently get return loads.

The general tendencies in motor-truck loading are shown by the average net and gross weights for trucks of each capacity as recorded in Table 3.

There is a tendency to load the 2, 2½, 3½, 5, and 6-ton trucks nearer the limit of their rated capacities. Trucks of these capacities can therefore be expected to be the principal motor trucks carrying loads in excess of rated capacity, since the ratio of their average net load to their capacity is higher than in the case of the other capacities.

TABLE 3.—*Average net and gross weight of trucks of the several capacities*

Capacity	Number of trucks	Average net weight	Average gross weight
		Pounds	*Pounds*
½ ton	5,072	720	2,950
¾ ton	1,375	1,090	5,210
1 ton	5,167	1,440	4,370
1¼ tons	3,443	1,560	5,270
1½ tons	1,679	2,310	7,350
2 tons	4,435	3,570	10,040
2½ tons	1,859	4,660	11,580
3 tons	335	4,430	12,770
3½ tons	3,507	6,020	15,820
4 tons	148	6,690	16,820
5 tons	6,897	8,680	20,170
5½ tons	236	8,440	20,380
6 tons	16	10,900	22,200
6½ tons	430	10,180	22,590
7½ tons	218	9,520	22,200

Almost one-third of the loaded motor trucks observed on the Connecticut highways during the survey carried net loads in excess of their rated capacities. Of the total number of loaded trucks, 30.2 per cent were loaded over their rated capacity and 2.1 per cent had gross weights in excess of 25,000 pounds. Of all sizes of trucks, those of 2, 2½, 3½, and 5-ton capacity were found most frequently to be loaded beyond their rated capacity. The percentages of trucks of these sizes so overloaded were 38.1, 41.8, 38.1, and 41.1 per cent, respectively.

The fact that 2.1 per cent of the total gross loads exceed the 25,000-pound legal limit is significant. No gross loads over 25,000 pounds were found on trucks of 3-ton capacity or smaller. It would be almost impossible for trucks of these smaller sizes to carry a 25,000-pound gross load. For this reason it would probably be more accurate to compute the percentage of loads in excess of the gross-weight limit on the basis of the number of loaded 3½ to 7½-ton trucks. Applying this method it is found that of the total number of 3½ to 7½-ton loaded trucks recorded, 6.4 per cent were loaded in excess of 25,000 pounds gross. On the terminal and class A highways overloaded trucks are more frequent. The highest percentage of trucks loaded beyond their rated capacity was found on the Boston Post Road.

TYPE OF TIRE EQUIPMENT

Pneumatic tires are used on 99 per cent of the ½-ton trucks. They also form the chief tire equipment for the ¾, 1, 1¼, and 1½-ton trucks. The trucks of larger capacity are mainly equipped with solid tires on both front and rear wheels, although there is some usage of combinations of the two tire types, using the solid type on the rear wheels and the pneumatic type on the front wheels.

The average net and gross loads of trucks vary considerably for trucks of the same capacity according to the type of tire equipment. Trucks equipped with pneumatic tires on both front and rear wheels are, in general, less heavily loaded than those which are equipped either with solid tires only or with a combination of pneumatic and solid tires. The heaviest loading occurs on the trucks that have solid tires on all four wheels.

Analyzing the trip mileage of the trucks with respect to the tire equipment, it appears that the majority of pneumatic-tired trucks—84.5 per cent—travel less than 40 miles. Of the solid-tired trucks, 58.8 per cent travel less than 40 miles; and of all trucks with the three types of tire equipment, 3.5 per cent of the pneumatic tired, 12.9 per cent of the solid tired, and 24 per cent of those equipped with a combination of solid and pneumatic tires, travel 100 miles or more at a trip.

LOADS PER INCH WIDTH OF TIRE

In order to determine the extent of violation of the statute prohibiting loads in excess of 800 pounds per inch width of solid tire and also providing that no commercial vehicle shall be so loaded that the weight on one axle is less than 20 per cent of the gross weight, an analysis was made of the weights of 4,580 loaded trucks. Excess loads per inch of width for the front axle were found to be exceptional. Of the 4,580 trucks analyzed, 102, or 2.2 per cent, were found to have loads in excess of 800 pounds per inch width of tire on the rear axle.

Examining the extent to which the overloading indicated by violation of this statute is confirmed by the other determinants of overloading—that is, loading in excess of rated capacity and loading in excess of 25,000 pounds gross weight—it is found that of the 4,580 trucks referred to above 2,810, or 61.4 per cent, were loaded to less than their rated capacity and of these only 20, or 0.7 per cent, were loaded in excess of 800 pounds per inch width of tire, channel measurement. Of the 1,770 trucks, 38.6 per cent of the total number, that were loaded in excess of rated capacity, 82, or 4.6 per cent, were loaded in excess of 800 pounds per inch width of tire, channel measurement; and of the 144 trucks the gross weight of which exceeded 25,000 pounds, 27, or 18.8 per cent, violated the tire-weight statute.

Data are also recorded and analyzed in the report as to the results secured by determining the width of tires by inside channel measurements and actual road contact measurements. The results indicate that a limitation of 800 pounds per inch of tire, channel measurement, is approximately equivalent to a limitation of 1,000 pounds per inch of width measured at the point of contact with the road surface.

Measurements were also taken of the thickness of solid truck tires. Very few trucks were encountered with tires less than 1 inch in thickness and the number operating on tires less than 1.5 inches in thickness varied from 2.8 per cent of the loads in excess of 25,000 pounds gross weight to 8.5 per cent of the trucks with less-than-rated-capacity loads. Over 60 per cent of all trucks operate on tires between 1.5 and 2.4 inches in thickness.

HIGHWAY UTILIZATION

The daily vehicle utilization of Connecticut highways is indicated by Figures 2 and 3. Net tonnage transported is the most reliable basis for the measurement of the service value of a highway for the transportation of commodities.

The 1,114 miles of improved highways in the State trunk-line system carry an average of 159,000 net ton-miles and 575,000 gross ton-miles of motor-truck traffic per day. During the year period—September, 1922, to September, 1923—these highways carried approximately 58,000,000 net ton-miles and approximately 210,000,000 gross ton-miles of truck traffic. During the same period the total vehicle utilization of the State system was approximately 414,000,000 vehicle-miles, of which 59,700,000 were truck-miles and 354,300,000 were passenger car- miles. The State trunk highway system includes 7.2 per cent of the total highway mileage in the State. The State-aid system includes 4.3 per cent and the town roads 88.5 per cent of the total mileage of the State. No accurate data regarding the vehicle-mileage utilization of the State-aid and town highway systems are available; but on the basis of scattered data it is estimated that approximately 60 per cent of the total vehicle mileage is on the State highway system, and that the total annual vehicle-mileage utilization of all highways in Connecticut is therefore approximately 690,000,000 vehicle-miles.

TABLE 4.—*Trip mileage of passenger cars used for business and nonbusiness purposes*

Trip mileage	All passenger cars	Business cars	Nonbusiness cars
	Per cent	*Per cent*	*Per cent*
0-19	55.7	65.3	48.0
20-39	16.3	14.4	17.9
40-59	9.2	8.2	10.1
60-79	5.0	3.9	5.9
80-99	3.6	1.7	5.1
100-149	4.3	3.3	5.1
150-199	1.7	1.2	2.1
200-299	2.7	1.5	3.7
300-399	.5	.2	.7
400-499	.2	.1	.4
500 and over	.8	.2	1.0
Total	100.0	100.0	100.0

Of the annual 354,300,000 passenger car-miles on the State system, 32.3 per cent, or approximately 114,400,000 passenger car-miles, represents the business use of passenger cars, and 67.7 per cent, or approximately 239,900,000 passenger car-miles, represents the nonbusiness use of passenger cars. The average trip mileage of passenger cars used for business purposes is considerably below the corresponding average for nonbusiness usage, as indicated by the analysis of the two kinds of usage presented in Table 4.

Of all passenger cars using the highways 44.5 per cent are business cars and 55.5 per cent nonbusiness cars. The variation between the proportion of business and nonbusiness usage on a vehicle basis and on a mileage basis is due to the shorter average trip mileage of business cars.

The percentages of all cars used for business and nonbusiness purposes, classified according to trip mileage, are shown in Table 5.

The number of passengers per car averages 2.7 persons for all passenger cars. For business cars the average is 1.8 persons, and for nonbusiness cars, 3.2 persons. The passenger mileage on the State highway system for the year period was 206,000,000 for business cars and 768,000,000 for nonbusiness cars, or a total of 974,000,000 passenger-miles.

TABLE 5.—*Percentages of all passenger cars used for business and nonbusiness purposes, classified according to trip mileage*

Trip mileage	Business cars	Nonbusiness cars
	Per cent	*Per cent*
0-19	52.2	47.8
20-39	39.3	60.7
40-59	39.4	60.6
60-79	34.8	65.2
80-99	21.6	78.4
100-149	34.5	65.5
150-199	30.9	69.1
200-299	24.6	75.4
300-399	17.6	82.4
400-499	17.2	82.8
500 and over	12.6	87.4
Total	44.5	55.5

STATE SYSTEM EARNS BIG RETURN ON INVESTMENT

Applying assumed values for saving in vehicle operation of 1 cent per passenger car-mile and 3 cents per truck-mile to the traffic on the 1,114 miles of the State trunk highway system, an estimated service value of the highway system can be obtained. As stated before the annual vehicle mileage on this system during the year period—September, 1922, to September, 1923—was approximately 414,000,000 vehicle-miles, of which 59,700,000 were truck-miles and 354,300,000 were passenger car-miles. On this basis of valuation the annual service value of the system is $1,791,000 for truck traffic and $3,543,000 for passenger car traffic, a total of $5,334,000 for the system.

A valuation based on motor-vehicle operating costs, however, does not represent the total service value produced by the highway system and the vehicles using the highways. Highway improvements increase real property values. Highway transportation adds time utility to the value of goods by the rapid movement at the time of demand and also produces place utility by making goods available for use by transporting them from the place of supply to the place of demand.

The value in 1923 of the 1,114 miles of improved highways on the State trunk-line system, including bridges but exclusive of right of way, is estimated by the Connecticut State Highway Commission at approximately $23,000,000. Based on the above estimate of the service value of the highway system to highway users, and without considering the additional service value indicated above, the State trunk highway system earns an annual return of approximately 23 per cent on the investment.

Vehicles of foreign registration form an important part of both motor-truck and passenger-car traffic on the State system. Foreign trucks comprised 10.9 per cent of all trucks recorded, but on account of their greater trip mileage (40 miles over Connecticut roads as compared with 14 miles for Connecticut trucks) the utilization on a ton-mileage basis is much higher. They also carry a greater tonnage than the Connecticut trucks. This is to be expected, since most of them are operating on the longer hauls where the larger capacities are generally used. On the basis of ton-mileage the foreign trucks furnish almost one-third of the total truck traffic.

Passenger cars of foreign registration were 21.1 per cent of all passenger cars recorded and furnished 43.4 per cent of the total passenger-car mileage.

TRANSPORTATION OF COMMODITIES

In 1923 there were 29,140 and in 1924 33,776 motor trucks registered in Connecticut. These motor trucks were in constant use carrying commodities over the highways and streets. During the period September, 1922, to September, 1923—the haulage of these commodities over the 1,114 miles of the State highway system amounted to approximately 58,000,000 ton-miles and the inclusion of other highways and city streets would substantially increase this amount.

Motor-truck transportation in the State is of three principal types.

1. Transportation within the market and trading area of a city or town, including—

(*a*) Distribution of commodities from wholesaler to retailer, and from retailer to consumer.

(*b*) Marketing of commodities produced in the area, and

(*c*) Pick-up and delivery service between railroad or water terminals and shipper and consignee.

The movement is purely local, the major part of it being within the city or town. It uses the rural highways only in so far as the market or trade area of the city extends beyond its political limits.

2. Complete transportation from the shipper's place of business to the consignee's place of business for comparatively short-haul shipments. This type of transportation is interurban rather than local or suburban in character and is generally limited to distances of less than 40 miles.

3. Comparatively long-distance transportation, including—

(*a*) The transportation of specialized commodities, notably those which require a special preparation for shipment by rail or water that can be avoided by motor-truck shipment; and those which, because of their high value or their perishability, require rapid-delivery service; and

(*b*) Emergency transportation during periods of congestion and embargoes on other transportation facilities.

From Table 6 it is evident that of the total net tonnage transported over the State highway system more than one-third is moved distances of less than 10 miles, approximately two-thirds is moved distances of less than 30 miles, and only 15.2 per cent is moved distances of 60 miles and over. The table includes commodities transported over the State system only and excludes the large volume of motor-truck tonnage which is transported over city streets.

TABLE 6.—*Distribution of net tonnage of commodities transported by motor truck over the Connecticut State highway system by length of haul*

Length of haul (miles)	Proportion of total net tonnage	Length of haul (miles)	Proportion of total net tonnage
	Per cent		*Per cent*
0–9	36.3	70–79	2.6
10–19	19.2	80–89	.8
20–29	11.6	90–99	1.0
30–39	9.1	100 and over	8.6
40–49	4.0		
50–59	4.6	Total	100.0
60–69	2.2		

Approximately 350 distinct commodities were recorded on trucks using the State highway system during the period of observation, but of the total net tonnage 69 per cent was made up of manufactured goods. The distribution by types of commodities is shown in Table 7.

TABLE 7.—*Principal commodities transported by motor truck on State highway system*

Commodity	Percentage of total net tonnage	Commodity	Percentage of total net tonnage
Groceries	6.8	Ice cream	2.0
Gravel, sand, and crushed rock	5.3	Iron and steel bar	1.8
General express	4.8	Brick	1.5
Gasoline	4.5	Feed and grain	1.5
Household goods (used)	4.5	Furniture (new)	1.4
Coal	3.3	Vegetables	1.3
Lumber	3.3	Paper	1.2
Bread and bakery goods	3.0	Wire	1.1
Meat, fresh	2.9	Ice	1.0
Milk, fresh	2.7	Tires, rubber	1.0
Fruits	2.7	Wood, cord, and kindling	.9
Brass, copper, and lead	2.5	Miscellaneous commodities	34.4
Beverages	2.4	Total	100.0
Textiles	2.2		

The length of haul and the average unit load of the different commodities varies considerably. Of the materials hauled 9 miles and less, gravel, sand, and stone are first in importance; in the 10 to 19 mile haul they rank third in importance, and for longer hauls they are not among the important commodities. Coal appears as an important commodity only in the shortest haul groups, groceries in all haul groups up to 70 miles, and gasoline among the commodities hauled all distances up to 30 miles. The commodities of greatest importance in the long-haul groups are household goods (used), textiles, rubber tires, and crude rubber. Household goods increase in relative importance with increase in the distance hauled, constituting 5.1 per cent of the movement from 40 to 49 miles and 23.5 per cent of the haulage for distances of 100 miles and more. Rubber tires and crude rubber appear among the important commodities only for hauls of 100 miles or more, indicating the very specialized character of the movement over such long distances.

The movement of foodstuffs, consisting largely of retail distribution and the marketing of such products as milk, is predominantly a small-truck movement. Household goods are transported chiefly in trucks of 1½ to 4 tons capacity, and constitute 11.7 per cent of all goods transported in 2½-ton trucks. Gasoline is hauled largely in trucks of from 2 to 4 tons capacity, and gravel, sand, stone, brick, and cement are hauled largely in trucks of 3½-ton and larger capacities. Textiles, metal products, and paper (largely news print) are also hauled in large-capacity trucks.

TRUCK AND RAIL TONNAGE COMPARED

Considerable data were collected as to the character and volume of freight transported by highway and railroad between selected points both within and outside of the State. The points selected represent distances varying from 17 to 140 miles, and all have direct connection by rail and highway. Between New Haven and Bridgeport 75.2 per cent of all the freight moves by motor truck, 0.9 per cent as l. c. l. rail shipment, and 23.9 as car-load rail shipment. The distance is 17 miles by rail and 18 miles by highway. Between New York City and Waterbury, a distance of 93 miles by highway and 88 miles by rail, the figures are 8.7 per cent by motor truck, 17.5 per cent l. c. l. freight, and 73.8 per cent car-load freight.

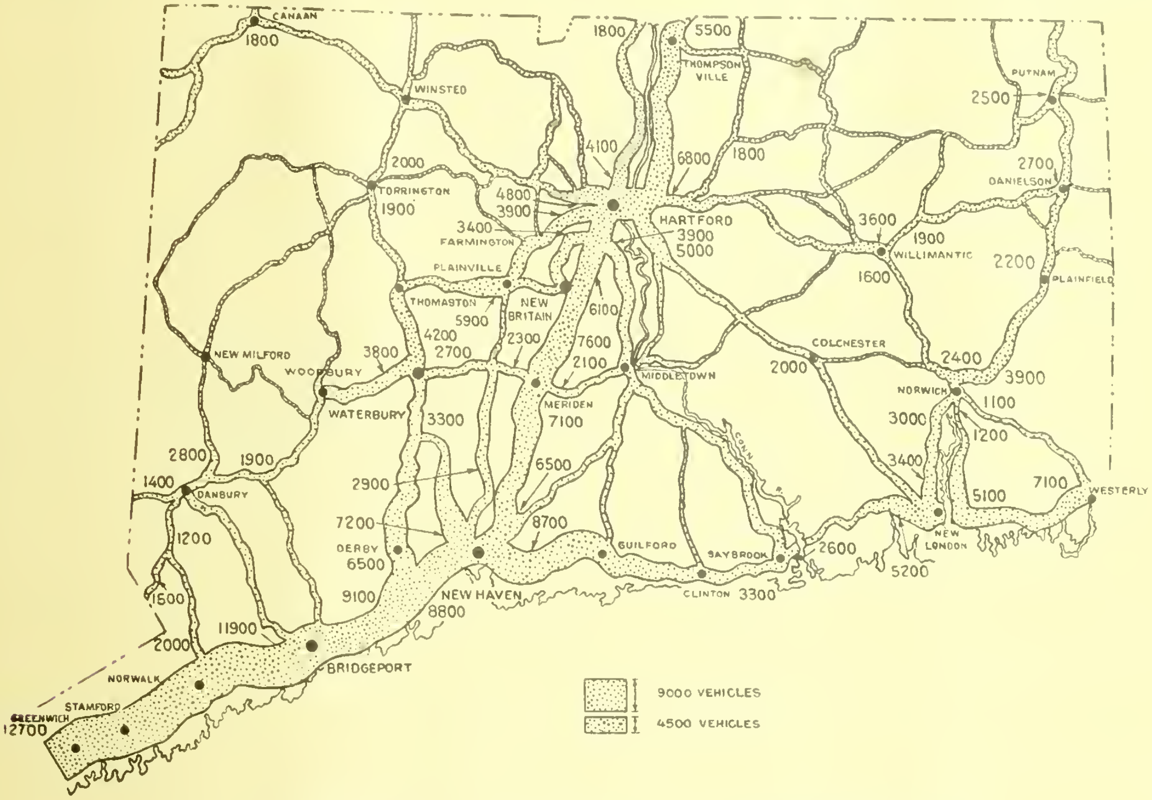

Fig. 6.—Estimated Total Traffic Density on State Highways in 1930

As motor-truck freight is primarily a package, or small unit shipment, it is more strictly comparable with rail l. c. l. freight than with all rail freight. Of all package freight the motor truck carries over 90 per cent between points less than 60 miles apart. Between points over 60 miles apart the percentage varies considerably, but the tendency toward relatively decreasing truck tonnage with the increase in distance is very apparent.

The importance of the motor truck in the transportation of products of agriculture and products of animals is even more apparent. These classifications include a large part of the movement of perishable foodstuffs. For products of animals the motor-truck tonnage greatly exceeds the rail l. c. l. tonnage between all points analyzed except New York City and Springfield, a distance of approximately 134 miles, and exceeds the total rail tonnage between all points except New York City and Springfield, and New York City and New Haven. The large car-lot movement of these products between New York City and New Haven is largely accounted for by the fact that New Haven is the wholesale distribution market for a considerable area.

Of manufactures the motor-truck tonnage exceeds the rail l. c. l. tonnage between all points analyzed except New York City and Springfield, and New York City and Waterbury; but the total rail tonnage exceeds that of the motor truck for all distances over 60 miles and also between Hartford and Springfield, a distance of approximately 25 miles. Between the latter points, however, 3,312 tons of the total of 3,627.8 tons, or over 90 per cent of the carload tonnage of manufactures, consists of refined petroleum and its products.

The importance of the motor truck in the transportation of commodities is clearly indicated, and the data warrant the following conclusions:

1. A very considerable part of the package freight between origins and destinations in the Connecticut territory less than 50 miles apart is transported by motor truck.
2. As distance between origin and destination increases transportation by motor truck becomes of less importance.
3. The ratio of motor-truck freight to rail freight between two points varies greatly with distribution and marketing practices. Cities which are distributing points for surrounding areas receive a larger amount of freight in bulk lots, such as rail carload shipments, than cities which distribute to the local market alone.
4. For origins and destinations between which the exchange of freight includes bulk commodities, such as coal and oil, the

tonnage transported by motor truck compared with total rail tonnage is relatively small. As such commodities are a necessary part of the freight into nearly all cities, motor-truck freight becomes a relatively small part of total freight.

FORECAST OF TRAFFIC IN 1930

A knowledge of the trend of traffic development on a highway system is a prerequisite to the establishment of an adequate and scientific plan of highway improvement. The building of a highway which will not meet traffic demands during the expected life of the improvement is a poor investment, resulting in traffic congestion and early reconstruction. On the other hand, the building of a highway with a traffic capacity in excess of the need that may be expected to develop during the life of the improvement is also uneconomic, since it involves an outlay of funds which could more advantageously be used for other highway improvements. A knowledge of future traffic is therefore essential to the establishment of a sound plan of highway improvement.

The most scientific method of future traffic prediction is by projecting past traffic trends. This method has been found accurate in the prediction of population, business conditions, railway traffic, and other economic factors. Accurate prediction on the basis of past trends is possible only when the trend over a considerable period of years is known and also when the period is one of normal development.

While there are no adequate records of traffic development in the State, there are complete records of the motor-vehicle registration for a series of years, and the experience of other States demonstrates that these may be employed as a basis for traffic prediction, since it is found that there is a close and practically constant relation between the rates of growth of registration and traffic. The States in which the relation between the two factors has been determined are Maryland, Maine, and Wisconsin. In each of these States both traffic and registration records are available for a number of years.

An analysis of the data from these States indicates clearly that traffic and registration may be expected to increase at approximately equal rates and that the prediction as to registration may be taken as the basis for the prediction of traffic.

It may not be assumed, of course, that the future traffic will be distributed in exactly the same proportions as the present traffic on the various roads. The development of new highways, unusual industrial or resort developments, and suburban expansion will affect the traffic in smaller areas.

The number of persons per vehicle in Connecticut for each year since 1917 is easily determined from available registration statistics and has been plotted graphically. To this curve there has been fitted a smooth curve using the method of least squares, and the latter has been extended to indicate the persons per vehicle in 1930. Table 8 shows the estimated persons per car determined in this manner, the population estimated by methods used by the Bureau of the Census and the resulting registration calculated from these two sets of figures for each year between 1925 and 1930.

Table 8.—*Estimate of persons per car, population and motor-vehicle registration in Connecticut, 1925 to 1930*

Year	Estimated persons per car	Estimated population	Estimated motor-vehicle registration
1925	6.22	1,531,250	[1] 246,000
1926	5.47	1,558,640	285,000
1927	4.80	1,586,030	330,000
1928	4.22	1,613,410	382,000
1929	3.70	1,640,800	443,000
1930	3.25	1,688,180	513,000

[1] The actual motor vehicle registration in 1925 was 250,647 vehicles, varying from the estimated registration by less than 2 per cent.

By comparing the registration of 1923 with the estimated 1930 registration, as above, a factor of increase is determined which when applied to the observed traffic of 1923, assuming traffic and registration to increase in the same proportion, indicates the probable 1930 traffic on the principal roads as shown in Figure 6.

Assuming no change in the present State highway system between 1923 and 1930, the traffic upon these highways is expected to be as shown. The predicted density on the present heavy-traffic routes indicates the pressing need for a comprehensive improvement program during this period.

The Post Road may be expected to carry a daily average of 12,700 vehicles at the New York line, and an average of approximately 9,000 vehicles per day on the section between Greenwich and New Haven; the same route from New Haven to the Massachusetts line may be expected to carry over 6,000 vehicles per day over the greater part of the route.

If the present ratio between average traffic and maximum traffic continues until 1930, approximately two and one-half times the predicted average traffic may be expected to attempt to use these routes on a Sunday during the month of maximum traffic.

The tremendous volume of traffic that may be expected in 1930 on the Post Road and other heavy-traffic routes will require extensive enlargements of the traffic capacity of these routes or the opening of alternate routes in order that adequate highway service may be provided.

Improvements on routes of secondary traffic importance will also be required, particularly routes at present improved with the lower types of surfaces, but the present heavy-traffic routes will continue to be the most important problem in the provision of adequate and economical service to the users of the Connecticut highway system.

ADAPTATION OF ATTERBERG PLASTICITY TESTS FOR SUBGRADE SOILS

Reported by A. M. WINTERMEYER, Bureau of Public Roads

A NUMBER of soil tests have been investigated recently by the Bureau of Public Roads in an effort to develop a simple test which would indicate the effect of moisture on the supporting value of the soil or at least that moisture content beyond which a soil has little, if any, definite supporting value. Among the methods investigated has been the Atterberg soil plasticity test, and the results obtained to date with this test have been so satisfactory that it has been decided to employ it regularly in conjunction with the other tests now being used by the bureau in the surveys which are being made to determine the influence of subgrades, subbases and drainage on the condition of pavements. A detailed description of the procedure used in these surveys will be given in forthcoming articles in PUBLIC ROADS.

In the absence of definite knowledge as to the specific properties of soils which influence their behavior as road subgrades it has been assumed that volumetric changes and supporting value are of prime importance and the principal effort has been directed toward the development of suitable tests for determining these properties, an important consideration being the adaptability of the tests to field use. It is in this respect that the laboratory tests for bearing power, moisture equivalent, and volumetric change have been unsatisfactory. They are not lacking in consistency of result or possibility of interpretation; but they require the use of equipment and instruments which can not be employed under field conditions, and the relatively long time necessary for their performance also militates against their use.

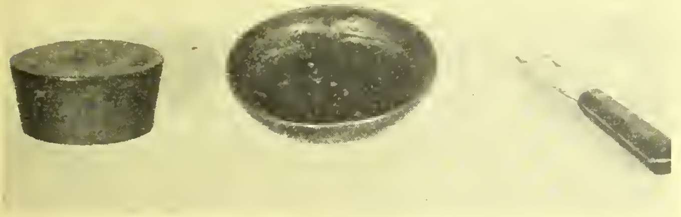

FIG. 1.—RUBBER STOPPER, BRASS DISH, SPATULA AND GLASS PLATE USED IN MAKING LOWER PLASTIC LIMIT AND LOWER LIQUID LIMIT TESTS

It is these facts which give importance to the field methods for determining moisture equivalent and lineal shrinkage [1] recently developed by the Bureau of Public Roads and lend interest to the plasticity test described by Albert Atterberg in the International Reports on Pedology, 1911.

Although plasticity has been defined by many authors, and is attributed by each to various factors, no authoritative definition has yet been given. We may say that a soil is slightly or highly plastic but such a characterization is vague in that the degree of plasticity is not definitely stated. We know, however, that the plasticity of a soil varies with the quantity of water added, increasing with the percentage of water up to a certain point beyond which any further addition of water reduces the plasticity and the soil assumes a liquid form. Passing over the many factors which contribute to the plasticity of a soil, its application to subgrades concerns the two extreme points designated as the *lower plastic limit*, or that moisture content at which the soil becomes plastic, and the *lower liquid limit*, or that moisture content at which the soil passes from the plastic into the liquid state.

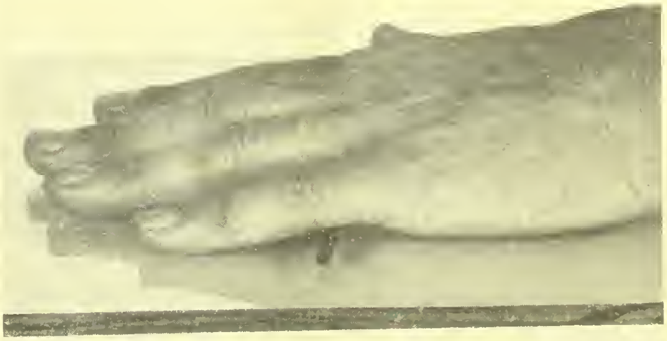

FIG. 2.—ROLLING PHASE OF THE LOWER PLASTIC LIMIT TEST

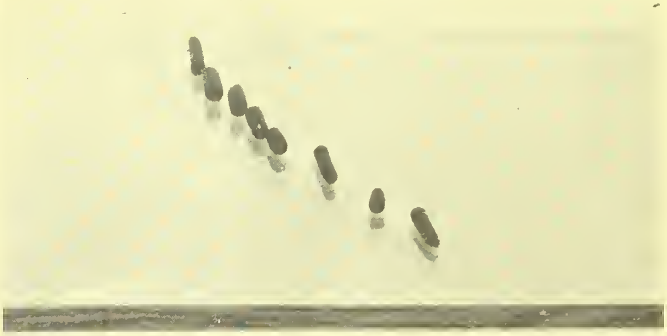

FIG. 3.—END POINT OR CRUMBLING PHASE OF LOWER PLASTIC LIMIT TEST

THE ATTERBERG PLASTICITY METHOD

The apparatus required for the Atterberg tests, as made by the Bureau of Public Roads, includes a drying oven such as is usually found in all laboratories; a glass plate or some other smooth surface such as glazed paper upon which to roll the soil; a rubber stopper (preferably No. 12); a 3-inch spatula, and a brass dish about 3 inches in diameter and 1 inch in depth as shown in Figure 1. An evaporating dish may be used but there is always the danger of breakage and painful cuts resulting from its substitution.

To determine the lower plastic limit a sample of the soil, which has been passed through a 10-mesh or 2-millimeter sieve, is mixed with water until it becomes plastic. It is then rolled with the palm of the hand on the glass plate or other smooth surface into threads or rolls about one-eighth inch in diameter as shown in Figure 2. The threads thus formed are then worked together and again rolled out as before; and this process is repeated until the soil crumbles instead of forming a thread, as illustrated by Figure 3. When this condition is reached the moisture content of the soil is determined and expressed as a percentage of the dry weight of the soil which is known as the lower plastic limit. The limiting condition is very easily recognized and no difficulty is experienced in closely checking the results.

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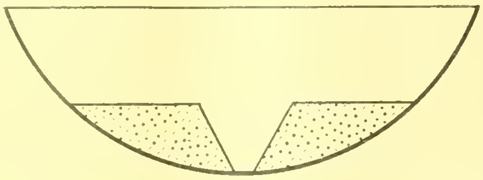
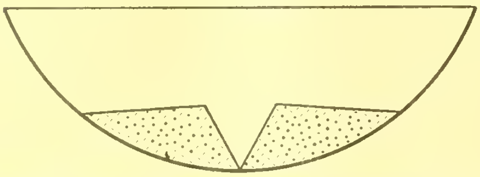
[1] Practical field tests for subgrade soils, by A. C. Rose, PUBLIC ROADS, vol. 5, No. 6, Aug. 1924.

In order to determine the lower liquid limit a small quantity of soil is placed in the brass dish and mixed with water until it becomes pasty. It is then shaped into a smooth layer approximately three-eighths inch thick at the center, and divided into two portions with a spatula as indicated by Figure 4, *A*, so that the space separating the two portions at the bottom is about one-eighth inch. One side of the dish is then rapped sharply with a glancing blow 10 times against the rubber stopper held in one hand, after which the other side is similarly rapped the same number of times, care being taken to make the impacts as nearly as possible of the same intensity and not too vigorous. The object of this procedure is to cause the two portions of soil to join at the bottom, as shown in Figure 4, *B*, and when the moisture content is just sufficient to cause such a running together with the stipulated amount of rapping the lower liquid limit is said to have been reached. The appearance of the soil at the beginning and the end of the test is shown by Figures 5 and 6, respectively.



A.—SOIL SEPARATED BY A DISTANCE OF ABOUT ONE-EIGHTH INCH AT THE BOTTOM OF THE DISH BEFORE THE TEST



B.—BASE OF SOIL SECTIONS JOINED AT CONCLUSION OF TEST

Fig. 4.—Condition of Soil Sample Before and after Determination of the Lower Liquid Limit by the Atterberg Method

If after the prescribed amount of rapping the two portions of the soil do not run together and adhere more water is added and thoroughly incorporated by mixing; the divided pat is reformed and rapped as described; and this entire process is repeated, adding a little water each time, until the desired end is attained. The moisture content of the soil is then determined and expressed as a percentage of the dry weight of the soil which is the lower liquid limit.

To determine definitely whether the two portions are actually joined the spatula may be used to push one away from the other. If they separate along the line of division the end has not been reached and the test must be repeated with more water. The water should be added a small quantity at a time in order to avoid passing the critical point. If there is reason to believe that too much water has been added dry soil instead of water may be added until the proper end point has been reached.

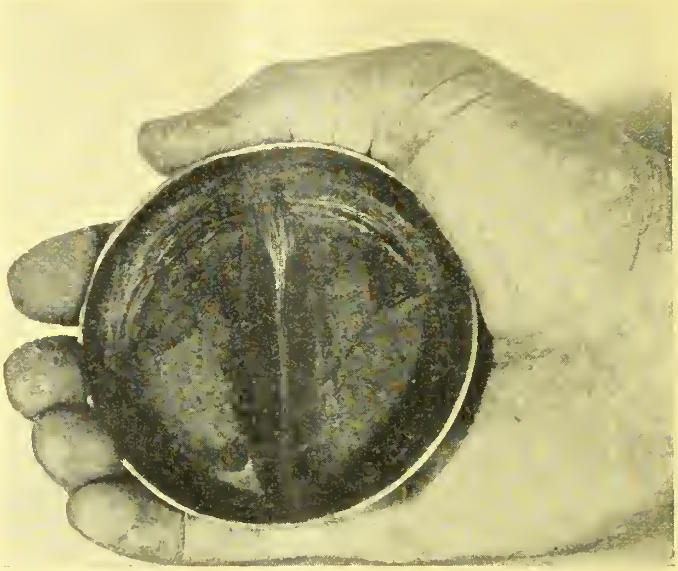


Fig. 5.—Base of Soil at Beginning of Lower Liquid Test Separated by a Distance of One-eighth Inch

The operator will soon learn to recognize the approach of the lower liquid limit by the ease with which the spatula will cut through the pat. This observation will serve as a guide and a warning that the desired end has been nearly reached and that any further water required must be added in very small quantities, preferably only a drop at a time.

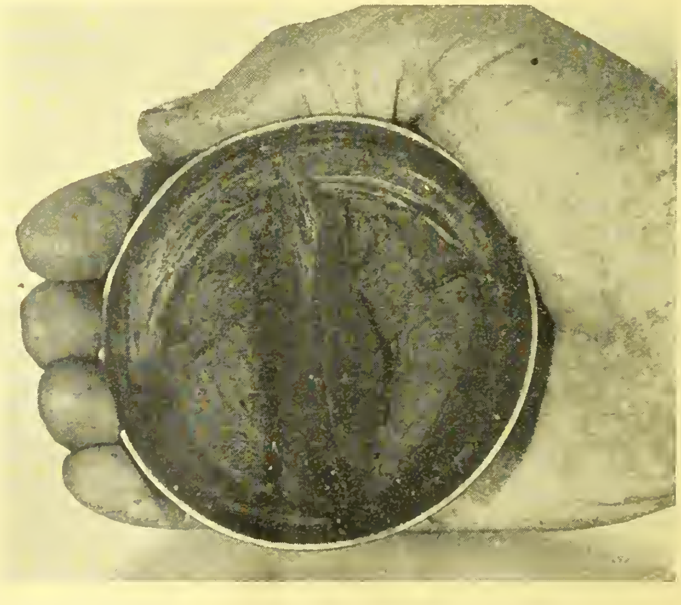


Fig. 6.—Condition of Soil at the Lower Liquid Limit. Two Portions of Soil Run Together and Adhere at Bottom

TESTS OF SANDY SOILS MOST DIFFICULT

The testing of sandy soils is more difficult and requires greater skill than the clay soils, and certain changes in the procedure described above are desirable. If the soil is friable or granular it is not advisable to strike the dish a glancing blow at the side. Instead it should be struck perpendicularly at the bottom. Soils of this class do not adhere firmly to the dish, and when struck at the side both portions will slip.

In conducting both tests certain precautions must be taken; otherwise close checks can not be expected. It is necessary that the soil and water be thoroughly mixed, especially in determining the lower liquid limit. It is extremely difficult to effect a uniform distribution of moisture in clay soils, and these must be worked until the consistency of the soil appears the same throughout. Here the sense of feeling must be the guide. The author has made several moisture determinations on different parts of a pat of heavy clay soil which appeared to be mixed thoroughly, yet showed a variation nearly as high as 2 per cent.

The compacting of the soil in the dish is another important factor. It is not possible to define exactly the pressure that should be exerted with the spatula when pressing and shaping the pat. It should be just great enough to form the soil into a compact mass, and this pressure should be approximately the same on all soils tested. The necessity for this is obvious. It can be readily seen that a soil differently compacted will require different moisture contents to reach the lower liquid limit and the results will therefore differ widely. Recognizing the end point seems to be the most difficult problem, but experience makes it possible for independent operators to check within very close limits.

In determining the lower liquid limit two methods may be used. Plastic soils may be worked from the plastic up to the liquid state or back to the lower liquid limit from the fluid state. Friable soils may be worked from the semi solid state to the lower liquid limit or back to the lower liquid limit from the fluid state. It is easier to work from the fluid state down to the lower liquid limit, but there is the disadvantage of handling and mixing greater quantities of soil, and as a rule the end point is passed and more water must be added to reach the end point.

THE MEANING OF THE PLASTIC RANGE

The difference between the lower plastic limit and the lower liquid limit, as determined by these tests, is what is called the *coefficient of plasticity* or *plastic range* of the soil, and the greater this range the more plastic is the soil. The difference between the two percentages expresses the range of moisture content within which the soil is in a plastic state.

Soils may be divided into two classes designated as plastic and friable, and the degree of plasticity or friability is indicated by the plastic range, the greater ranges indicating the more plastic soils. As we add water in increasing quantities to a plastic soil it passes from the solid or semisolid state into the plastic state and then into the liquid state. The friable soils, on the other hand, pass directly from the solid to the liquid state. The coefficient of plasticity or plastic range of plastic soils may vary from less than 2 in the more sandy or silty soils to as high as 35 or more in the heavier clays; while that of the friable soils is zero, i. e., they have no plastic range.

The silty friable soils differ from the sandy friable soils in that they require a greater percentage of moisture before they reach the point of transition to the liquid state. Quite frequently soils are encountered that appear sandy yet have a high plastic range.

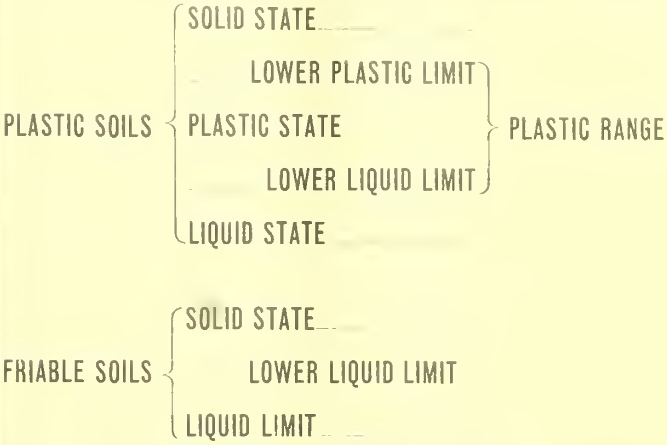


FIG. 7. PLASTIC SOILS HAVE A PLASTIC RANGE EXTENDING FROM THE LOWER PLASTIC LIMIT TO THE LOWER LIQUID LIMIT. FRIABLE SOILS HAVE ONLY A LOWER LIQUID LIMIT

According to Kinnison [2] the shape of the sand particles has a much greater influence on the plasticity than the size, and this observation may explain such apparently anomalous cases. The different transition stages which distinguish the plastic from the friable soils are shown diagrammatically in Figure 7.

TABLE 1.—*Original and check results of the plasticity tests on plastic soils by an experienced operator*

State	Soil sample number	Lower plastic limit		Lower liquid limit	
		First test	Second test	First test	Second test
		Per cent	*Per cent*	*Per cent*	*Per cent*
Minnesota	1003	15. 7	15. 5	17. 4	17. 6
Maryland	1711	18. 9	18. 4	22. 3	22. 1
Ohio	1459	19. 8	21. 2	22. 9	22. 8
Virginia	1838	20. 6	20. 8	22. 9	23. 2
Do	1835	21. 3	20. 9	25. 5	25. 7
Do	1841	22. 8	22. 9	27. 2	27. 1
Do	1833	23. 5	23. 6	30. 6	30. 8
Do	1832	25. 0	25. 1	31. 2	31. 3
South Dakota	1284	26. 3	26. 1	45. 2	45. 3
Do	1282	26. 6	27. 7	49. 0	49. 0
Virginia	1987	27. 5	27. 8	40. 4	40. 8
Do	1837	29. 1	29. 0	34. 0	33. 8
Do	1650	30. 2	29. 5	44. 5	44. 8
South Dakota	1283	32. 3	29. 2	50. 8	51. 3
Texas	1696	32. 5	32. 1	55. 2	53. 3
North Carolina	1619	27. 3	27. 9	50. 7	50. 7
Texas	1699	33. 3	33. 4	68. 0	67. 8
Do	1686	35. 6	33. 4	58. 0	58. 0
South Dakota	1273	36. 5	35. 7	53. 7	55. 4
Texas	1698	36. 2	36. 1	68. 6	67. 8
South Dakota	1274	44. 2	42. 8	60. 1	60. 9
Do	1276	43. 5	43. 5	57. 8	57. 8

CONSISTENCY OF TEST RESULTS PROVED BY BUREAU EXPERIENCE

The experience of the bureau with the Atterberg tests indicates that an experienced operator making duplicate tests may be expected to check his results very closely. This is shown by the check results, representative of over 500 soils tested to date, given, for the plastic soils in Table 1, and, for the friable soils, in Table 2. The proximity of all points in Figures 8, *A* and *B*, further emphasizes the consistent agreement between duplicate tests. These curves illustrate the consistency of the tests in the hands of experienced operators.

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[2] A study of the Atterberg Plasticity Method, by Chas. S. Kinnison, Bureau of Standards, Technologic Paper No. 46, May 25, 1915.

In order to ascertain the maximum variation that may be expected two series of tests have been made on the same soils at different times and with different operators, one experienced and the other inexperienced.

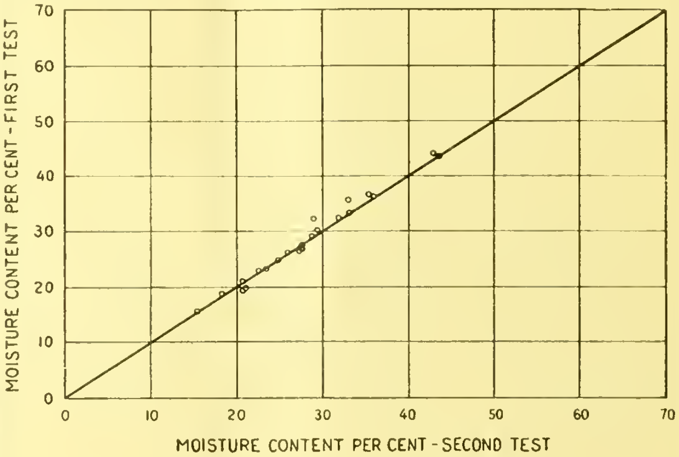
Table 2.—*Original and check results of the plasticity tests on friable soils by an experienced operator*

State	Soil sample number	Lower liquid limit	
		First test	Second test
		Per cent	*Per cent*
South Carolina	1563	15.3	14.4
Virginia	1766	16.6	16.2
Do	1763	18.0	18.5
Do	1762	19.8	19.0
Do	1783	20.5	20.3
Do	1774	20.3	21.4
Do	1772	22.4	22.4
Do	1784	23.6	23.3
Do	1856	23.8	23.8
Do	1769	24.8	24.6
Do	1874	27.8	27.7
Do	1776	30.7	31.6
Minnesota	37	32.1	31.9
Virginia	1915	37.1	37.1
Do	1442	41.2	43.4
Do	1649	44.9	45.0

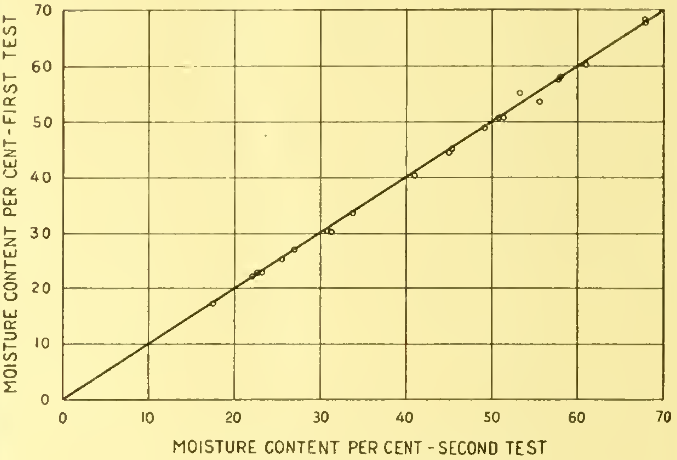
Table 3.—*Original and check results of the plasticity tests made by different operators and at different times*

State	Soil sample number	Lower plastic limit	
		First test	Second test
		Per cent	*Per cent*
Virginia	504	11.7	13.0
Do	499	12.2	13.6
Do	492	----	----
Do	502	12.8	15.8
Do	500	12.1	12.4
Do	496	13.3	15.5
Do	489	12.7	14.2
Do	486	15.1	19.0
Colorado	1632	15.9	17.5
Virginia	507	17.5	16.8
Colorado	1630	14.2	14.0
Do	1631	14.3	21.3
Do	1626	----	----
Do	1627	17.2	14.8
Do	1628	15.7	14.0
Do	1625	17.3	15.5
Illinois	1611	22.7	23.2
Colorado	1624	21.2	19.6
Ohio	1272	21.5	21.8
Colorado	1622	18.8	17.9
Utah	763	20.0	24.3
Colorado	1623	----	----
Utah	758	23.9	26.2
Do	759	23.5	24.0
Nevada	760	26.3	25.5
South Dakota	1281	26.4	27.3
Nevada	761	28.0	30.2
Utah	756	25.1	27.5
Do	762	24.2	24.9
Do	757	24.7	25.0
South Dakota	1283	30.8	23.1
Do	1277	36.1	33.0
Louisiana	1477	25.3	21.4
South Dakota	1273	34.2	31.2
Minnesota	924	28.1	28.1
South Dakota	1274	43.5	44.2
Do	1276	43.5	41.7

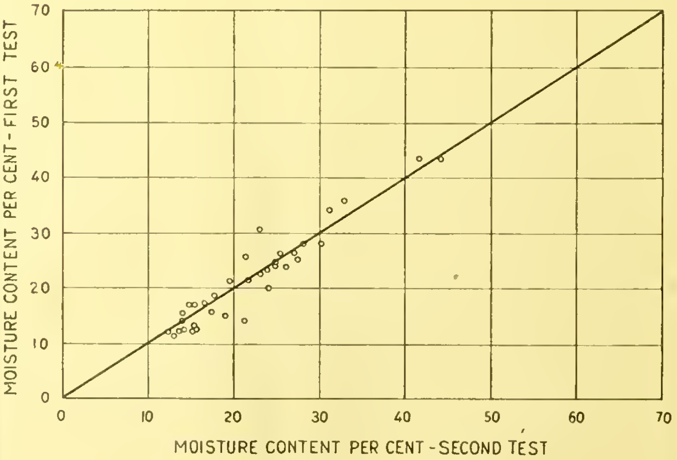
Results of these check tests for a wide range of soils, from different States, are shown in Table 3 and by the curve in Figure 8, *C*. These data indicate the degree of accuracy which might be obtained in tests of the same soil types when carried on in different laboratories and by different operators. Although the agreement between the first and second tests might not be as close as some would expect, it is probable that the agreement is closer than would be expected from tests of other engineering materials (such as cement or concrete) carried on in a similar manner.



A.-LOWER PLASTIC LIMIT DETERMINATIONS BY AN EXPERIENCED OPERATOR



B.-LOWER LIQUID LIMIT DETERMINATIONS BY AN EXPERIENCED OPERATOR



C.-LOWER PLASTIC LIMIT DETERMINATIONS BY INEXPERIENCED OPERATORS

Fig. 8.—Comparison of Check Tests, Using the Atterberg Method, by the Bureau of Public Roads

FIFTH INTERNATIONAL ROAD CONGRESS OPENS AT MILAN, ITALY

UNITED STATES SENDS OFFICIAL DELEGATES

AT THE FIFTH International Road Congress, which opens at Milan, Italy, on September 6, the United States will be represented for the first time by official delegates. Acceptance of membership in the Permanent Association of International Road Congresses and authorization for the appropriation of expenses of delegates to the current meeting was provided by Congress during the closing days of the last session in Senate Joint Resolution 62, which was signed by the President on June 18.

Acting on the joint recommendation of the Secretaries of State, Agriculture, and Commerce, the President has appointed as official delegates to the Congress: Thomas H. MacDonald, chief United States Bureau of Public Roads; Paul D. Sargent, chief engineer State Highway Commission of Maine; John N. Mackall, chairman State Roads Commission of Maryland; H. H. Rice, vice president General Motors Corporation; Geo. Pyke Johnson, National Automobile Chamber of Commerce; H. C. MacLean, commercial attaché, Department of Commerce at Rome; and H. H. Kelly, assistant trade commissioner, Department of Commerce at Paris.

The delegation will sail from New York on the steamship *President Harding* leaving August 25 and will arrive at Milan in time to take part in all meetings of the Congress which will be opened on September 6 and continue at Milan for six days, closing in Rome on the following day, September 13.

Simultaneously with the congress an international road exhibition will be held, under the patronage of the Communal and Provincial Administrations and the Chamber of Commerce of Milan, in the Fiera Campionaria. The exhibition will open on the 1st and continue until the 20th of September.

PROGRAM OF THE CONGRESS

The following program of the congress has been announced in circulars just received in the United States:

MONDAY, SEPTEMBER 6.
  Morning: 9 a. m.—Meeting of the International Permanent Commission (Castello Sforzesco). Opening of the offices for distribution of membership cards, etc. (Palace of Facolta Giuridica, Via S. Michele del Carso N. 25).
  10.30 a. m.—Official sitting of the congress (Castello Sforzesco).
  Afternoon: 2 p. m.—Sitting of the sections.
  Evening: Receptions.
TUESDAY, SEPTEMBER 7.
  Morning: 9 a. m.—Sitting of the sections.
  Afternoon: 2 p. m.—Sitting of the sections.
    4 p. m.—Visit to the exhibition and reception.
WEDNESDAY, SEPTEMBER 8.
  Morning: 9 a. m.—Sitting of the sections.
  Afternoon: 2 p. m.—Sitting of the sections.
    4 p. m.—Visits.
THURSDAY, SEPTEMBER 9.
  Visit to the Autodromo at Monza and the Autostrade. Milan Lakes.
  Lunch at Varese at the invitation of Ing. Gr. Uff. Piero Puricelli.
FRIDAY, SEPTEMBER 10.
  Morning: 11 a. m.—Sitting of the sections, with a view to arriving at conclusions.
  Afternoon: Visits or receptions.

EXCURSIONS

SATURDAY, SEPTEMBER 11.
  Group a. To Lake Maggiore; return to Milan the same evening.
  Group b.—To Lake Como; return to Milan the same evening.
SATURDAY AND SUNDAY, SEPTEMBER 11 AND 12.
  Group c.—Milan, Tirano, Stelvio, Merano, Trento.
  Group d.—Milan, Tirano, Aprica, Tonale, Trento.
MONDAY, SEPTEMBER 13.
  3 p. m.: Official closing of the congress in Rome and official reception in the Campidoglio.
TUESDAY, SEPTEMBER 14.
  9 a. m.: Archeological excursion.

SUBJECTS TO BE DISCUSSED

The program of subjects to be discussed by the congress, the names of the speakers for each subject, and the general speakers on each question are as follows:

FIRST SECTION

CONSTRUCTION AND UPKEEP

FIRST QUESTION, CONCRETE ROADS

Belgium.—(In collaboration): Messrs. E. Piens, chief engineer of bridges and roads, 33 Square Gutenberg, Brussels; Luyssen, engineer of bridges and roads, 33 Square Gutenberg, Brussels.

Denmark.—M. S. Ellert, chief engineer of bridges and roads of the county of Holback, Holback.

United States.—(In collaboration): Messrs. Clifford Older, M. Am. Soc. C. E., consulting engineer, 1739 Marquette Building, Chicago, Ill.; J. Shirley Bright, M. Am. Soc. C. E., construction engineer, U. S. Bureau of Public Roads, San Francisco, Calif.; Raymond W. Coburn, construction engineer, State Highway Department, Boston, Mass.; Charles R. Ege, Assoc. M. Am. Soc. C. E., manager highways bureau, Portland Cement Association, Chicago, Ill.; Herbert J. Kuelling, Assoc. M. Am. Soc. C. E., construction engineer, State Highway Department, Madison, Wis.; Robert M. Morton, Assoc. M. Am. Soc. C. E., State highway engineer, Sacramento, Calif.; Frank T. Sheets, M. Am. Soc. C. E., chief highway engineer, State Department of Public Works and Buildings Division of Highways, Springfield, Ill.; Leroy C. Smith, Assoc. M. Am. Soc. C. E., chief engineer, Wayne County Highway Commission, Detroit, Mich.; Charles M. Upham, Assoc. M. Am. Soc. C. E., formerly chief engineer, State Highway Department, Raleigh, N. C.

France.—Monsieur Féret, chief of the bridges and roads laboratory, Boulogne-sur-Mer.

Great Britain.—(In collaboration): A Harrison, esq., M. Inst. C. E., borough engineer and surveyor, Southwark, London S. E.; J. B. L. Meek, Esq., M. Inst. C. E., city engineer, Manchester; Richmond, esq., C. M. G., A. M., Inst. C. E., divisional road engineer, Ministry of Transport, London; W. P. Robinson, esq., county surveyor, Surrey County Council.

Italy.—M. Franklin Colamonico, inspecteur supérieur du génie civil, Ministry of Public Works, Rome.

Sweden.—Mr. Paul E. Wretlind, engineer of roads and bridges, director of the Limited Company "Wagförbättringer," Blasieholmsgatan, 3, Stockholm.

Low countries.—Dr. L. R. Wentholt, chief engineer of the Rijkswaterstaat (bridges and roads), Mariënburg, No. 80, Nimégue.

SECOND QUESTION, ROADS USING BITUMEN AND ASPHALT

Belgium.—Mr. Van Moere, chief engineer of roads and bridges, 1, rue Blondeau, Namur.

United States.—(In collaboration): Messrs. Charles S. Pope, M. Am. Soc. C. E., chief construction engineer, State highway department, Sacramento, Calif.; R. Keith Compton, M. Am. Soc. C. E., director of public works, Richmond, Va.; W. Herbert

Fulweiler, M. Am. Soc. T. M., chemical engineer, United Gas Improvement Co., Philadelphia, Pa.; Christian P. Jensen, M. Am. Soc. C. E., county surveyor, Fresno, Calif.; William H. Kershaw, Assoc. M. Am. Soc. C. E., manager the asphalt department, the Texas Co., Whitehall Building, New York City; Ralph E. Myers, M. Am. Soc. T. M., chemist New York State Highway Department, Albany, N. Y.; Philip P. Sharples, M. Am. Soc. T. M., consulting highway engineer, Redondo Beach, Calif.; Henry G. Shirley, M. Am. Soc. C. E., chairman State Highway Commission, Richmond, Va.; Francis P. Smith, M. Am. Soc. C. E., consulting highway engineer, 130 East Twenty-third Street, New York City

France.—(In collaboration): Messrs. Chauve, chief engineer of roads and bridges, Nice; Félix, engineer of roads and bridges, Versailles.

Great Britain.—W. J. Hadfield, esq., M. Inst. C. E., city surveyor, Sheffield.

Italy.—(In collaboration): Messrs. Giuseppe Sdralevich, chief engineer of the Technical Bureau of the Province of Milan; Ugo Conte, Technical Bureau of the Rome Municipality.

Sweden.—Mr. M. K. Joseph Gednt, engineer of roads and bridges, chief engineer of the "Nya Asfaltakiebolaget," Rörstrandsgatan, 3, Stockholm.

Low Countries.—Mr. M. B. J. Kerkof, member of the board of undertakings for the construction of modern routes, Blithoven (near Utrecht).

Switzerland. (In collaboration): Messrs. R. Schlaepfer, cantonal engineer, Herisau; Dr. P. Schlaepfer, professor at the Federal Polytechnic School, Zurich.

Third Question. Standardization of Trials for Acceptance of Materials for Roads

Denmark.—Mr. M. H. Rygner, chief engineer of the town of Odense, Odense.

United States.—(In collaboration): Messrs. Albert T. Goldbeck, Assoc. M. Am. Soc. C. E., formerly chief division of tests, U. S. Bureau of Public Roads, Washington, D. C.; Duff A. Abrams, M. Am. Soc. C. E., engineer of materials research laboratory, Lewis Institute, Chicago, Ill.; Harold F. Clemmer, Assoc. M. Am. Soc. C. E., engineer of materials, State Highway Department, Springfield, Ill.; Prevost Hubbard, M. Am. Soc. T. M., chemical engineer, the Asphalt Association, 25 West Forty-third Street, New York City; Claude L. McKesson, Assoc. M. Am. Soc. C. E., research engineer, California Highway Commission, Sacramento, Calif.; Horatio S. Mattimore, Assoc. M. Am. Soc. C. E., engineer of tests, State Highway Department, Harrisburg, Pa.

France.—(In collaboration): Messrs. Bouteville, engineer of bridges and roads, attached to the chief of the technical service of public ways and the lighting of the town of Paris, 34 Avenue Rollin, Paris; Vannenfville, engineer of bridges and roads, 98 Quai de la Râpée, Paris.

Great Britain.—M. W. J. Taylor, Esq., O. B. E., M. Inst. C. E., county surveyor, Southampton County Council.

Italy.—Mr. Luigi Torri, of the Technical Bureau of the Milan Municipality.

Monaco.—Dr. Guglielminetti, member of the Permanent International Commission of the Road Congress, Paris.

Switzerland.—(In collaboration): Messrs. R. Schlaepfer, cantonal engineer, Herisau; Dr. P. Schlaepfer, professor at the Federal Polytechnic School at Zurich.

Czechoslovakia.—M. Karel Spacek, engineer and professor at the Technical High School of Prague, Prague-Smichowulice Pavla Svandy, No. 7 n.

SECOND SECTION

CIRCULATION AND WORKING

Fourth Question. Census of Circulation

United States.—(In collaboration): Messrs. Henry R. Trumbower, economist, U. S. Bureau of Public Roads, Washington, D. C.; Thomas R. Agg, M. Am. Soc. C. E., professor of highway economics, Iowa State College, Ames, Iowa; J. Rowland Bibbins, consultant in transportation, Cosmos Club, Washington, D. C.; Arthur H. Blanchard, M. Am. Soc. C. E., professor of highway engineering and highway transportation, University of Michigan, Ann Arbor, Mich; William P. Eno, Eno Foundation, 1771 N Street, Washington, D. C.; Ernest P. Goodrich, M. Am. Soc. C. E., consulting engineer, 15 Park Row, New York City; Laurence I. Hewes, Ph. D., M. Am. Soc. C. E., deputy chief engineer, U. S. Bureau of Public Roads, San Francisco, Calif.; John N. Mackall, M. Am. Soc. C. E., chairman and chief engineer, State Roads Commission, Baltimore, Md.; William A. Van Duzer, M. Am. Soc. C. E., equipment and transport engineer, State Highway Department, Harrisburg, Pa.

France.—Mr. Delemer Léon, chief engineer of bridges and roads, secretary of the first section of the Board of Bridges and Roads, Ministry of Public Works, Paris.

Great Britain.—C. H. Bressey, Esq., C. B. E., F. S. I., chief engineer, Roads Department, Ministry of Transport, London.

Italy.—(In collaboration): Messrs. Ugo Cantalamessa, chief engineer of the Technical Bureau of the Province of Rome; Antonio Albertini, chief engineer of the Technical Bureau of the Province of Modena; Pietro Remondini, chief engineer of the Technical Bureau of the Province of Ravenna.

Low Countries.—(In collaboration): Messrs. G. J. Van den Broek, chief engineer of the "Rykswaterstaat," Surinamestraat, No. 1, The Hague; Dr. J. H. Van Zanten, municipal administrator of the town of Amsterdam, Statistics Office, Hotel de Ville.

Sweden.—Mr. Per Gustav Blidberg, lieutenant au Corps des Ponts et Chaussées, Märten Krakowsgatan, 5 Gothenburg.

Switzerland.—Mr. A. Altweg, cantonal engineer, St. Gallen.

Czechoslovakia.—Mr. Antonin Hlousek, engineer and chief adviser at the Ministry of Public Works, Libenskaulice, No. 453, Prague VII.

Fifth Question. Development and Organization of Towns in the Interest of Traffic

Belgium.—(In collaboration): Messrs. Maertens, managing director of the department of municipal thoroughfares, Agricultural Ministry and Public Works, 3 rue du Méridien, Brussels; Mullis, architectural engineer, chief of the bureau, department of municipal thoroughfares, Agricultural Ministry, 3 rue du Méridien, Brussels; Luyssen, engineer of bridges and roads, 33 Square Gutenberg, Brussels; E. d'Hoop, engineer, director of the technical department of the Brussels Tramways Co., 15 Avenue de la Toison-d'Or, Brussels.

Brazil.—(In collaboration): Messrs. Da Silva Freire, director of public works, Sao Paulo; J. Da Costa Ferreira, assistant director of the public ways, Municipal Prefecture, Rua D. Anna, 26 Rio de Janeiro.

United States.—(In collaboration): Messrs. Frederick L. Olmstead, landscape architect, member advisory committee on physical survey of regional plan of New York City and its environs, Brookline, Mass.; Charles B. Ball, M. Am. Soc. C. E., chief sanitary inspector, Department of Health, Chicago, Ill.; Harland Bartholomew, M. Am. Soc. C. E., engineer, City Planning Commission, St. Louis, Mo.; Frederic A. Delano, Hibbs Building, Washington, D. C.; Frederic H. Fay, M. Am. Soc. C. E., chairman Planning Board of the City of Boston, 200 Devonshire Street, Boston, Mass.; Morris Knowles, M. Am. Soc. C. E., consulting engineer, Pittsburgh, Pa.; Arthur A. Shurtleff, consultant, Metropolitan Planning Division, Commonwealth of Massachusetts and Boston Park Department, Boston, Mass.; Arthur S. Tuttle, M. Am. Soc. C. E., chief engineer, Board of Estimate and Apportionment, New York City; George S. Webster, M. Am. Soc. C. E., consulting engineer, 816 Widener Building, Philadelphia, Pa.; E. P. Goodrich, 15 Park Row, New York City.

France.—Mr. Massard, president of the second commission of the Municipal Board of Paris.

Great Britain.—(In collaboration): Chief Constable A. Bassom, O. B. E., New Scotland Yard, London; J. L. D. Elliot, Esq., C. B., New Scotland Yard, London; H. Vaughan Lanchester, Esq., F. R. I. B. A., 19, Bedford Square, London W. C. 1; G. L. Pepler, Esq., Ministry of Health, London.

Italy.—(In collaboration): Messrs. Massimo Settimi, Technical Bureau of the Rome Municipality; Tognetti, chief engineer at the Technical Bureau of the Municipality of Florence.

Low Countries.—(In collaboration): Messrs. L. N. Holsboer, director of public works of the town of Utrecht, 2 Nieuwe Graht, Utrecht; A. H. Sirks, chief of the municipal police of the town of Rotterdam, Central Police Bureau, Rotterdam.

Sweden.—Mr. Carl Gustav Bergman, captain of Bridges and Roads Corps, Riddarhustorget, 8 Stockholm.

Switzerland.—Mr. E. Bosshard, municipal engineer of the town of Zurich.

Czechoslovakia.—Mr. Posef Prada, chief adviser of the Police Board at Prague, Krokivaulice, No. 6, Prague Smichov.

Sixth Question. Roads Specially Reserved for Motor Cars

Belgium.—(In collaboration): Messrs. Cauterman, chief engineer of the Technical Department of the Province of East Flanders, Ghent; M. De Graer, engineer of the Technical Department of the Province of West Flanders, Furnes.

(Continued on p. 127)

PROCEEDINGS OF HIGHWAY RESEARCH BOARD

Reviewed by R. E. ROYALL, Assistant Highway Engineer, Bureau of Public Roads

THE widespread interest in highway research and the many problems involved which call for specialization in numerous lines, some related and some entirely dissimilar, are illustrated by the Proceedings of the Fifth Annual Meeting of the Highway Research Board of the National Research Council which has recently been issued. A glance through this publication shows that many agencies including the Federal and State Governments, and engineering and economic departments of universities and colleges are working on the various problems. The proceedings have been issued in two parts: Part I containing reports of the special committees; and Part II which is a report of the investigation of the economic value of reinforcement in concrete roads by C. A. Hogentogler.

FACTORS AFFECTING COST OF VEHICLE OPERATION INVESTIGATED

The report of the committee on economic theory of highway improvement contains a résumé of tests made to determine motor-vehicle wind resistance, tire wear, and gasoline consumption. Wind resistance measurements were made in a special wind tunnel at Manhattan, Kans., by L. E. Conrad. He finds that the equation for the wind resistance per square foot of projected area averaged for 14 makes of cars is, $P=0.00149\ V^{2.14}$, where P is the pressure per square foot in pounds and V is the speed in miles per hour.

Investigations of tire wear have been conducted at the University of Kansas by W. C. McNown and at the State College of Washington by H. J. Dana. In the former experiment a passenger car was operated over various types of surface at a speed of approximately 25 miles per hour and the loss in tire weight owing to wear was determined. The wear indices reported were as follows: Concrete and brick 1.0, Iowa gravel 2.2, chert gravel 7.3, and bituminous macadam (unsealed) 10.6.

In the tests at the State College of Washington, the tire wear in pounds per thousand miles was found to vary from 0.0477 for a light car to 0.1183 for a heavy car, on concrete and bituminous surfaces. For the light car on crushed basalt macadam the wear was 0.2972, whereas for the heavy car on water-worn gravel the wear was 0.6250.

The report on gasoline-consumption tests by Maj. Mark L. Ireland, of the Quartermaster Corps, shows rates of consumption under various conditions ranging from 6.7 to 17.7 miles per gallon for passenger cars and from 2.67 to 7.3 miles per gallon for trucks.

STRUCTURAL DESIGN OF ROADS

The report of the committee on structural design of roads contains some of the most important research developments of the year. Stresses in concrete roads have been investigated by actual field and laboratory tests by the Bureau of Public Roads and on the basis of mathematical theory of elasticity by H. M. Westergaard, of the University of Illinois. Professor Westergaard presents a method of calculating the required thickness of a concrete road slab for a given wheel load if the allowable stresses are assumed. Using the assumed stresses, tables, and diagrams presented in the report, computations may be quickly made which will indicate what additional thickness of concrete is required for an increase in wheel load and what may be saved in thickness by eliminating the heaviest vehicles. Information of this character is particularly valuable in apportioning the cost of highways to different classes of traffic.

The Bureau of Public Roads tests are reported by L. W. Teller, who describes the measurement of stresses in concrete roads, using a compact-recording strain gauge of high magnification. With this instrument it has been possible to study the maximum stress in pavement slabs, the balance of the various cross-sectional designs, the transfer of stress across molded joints and the effect of subgrade conditions. Tests with this instrument have been made in cooperation with the Pennsylvania State highway department and with the highway authorities of Cook County, Ill., while tests using 6-wheel trucks for loading have been conducted at the Arlington Experiment Farm.

W. K. Hatt of Purdue University presented a report on experiments on extensibility of concrete. These experiments were undertaken to determine the factors affecting extensibility, studying first the effect of curing conditions and mesh reinforcement. They have not yet been completed but have progressed to the point where certain well-defined indications can be stated.

Subgrade investigations for the year 1925 by the Bureau of Public Roads were reported by Ira B. Mullis. This report discusses the characteristics of good and poor subgrade soils, the movement of surface and ground water, treatment of wet cuts and of soils having a high capillary potential, and the influence of climate on subgrade conditions.

H. E. Breed, of New York University, presented a summary of tension tests of concrete briquettes reinforced with steel fabric. The object of the tests was to determine the benefit of steel fabric reinforcing during the first seven days after placing when concrete is weakest and has to resist the greatest contraction. Briquettes approximately four times the size of ordinary tension briquettes were used. Among the conclusions it is stated that concrete of the dense mix used, treated as described, reinforced with one-eighth to one-fourth per cent steel fabric, is benefited in tensile strength far more than the percentage of steel used would indicate.

W. D. Somervell, of the North Carolina Highway Commission, described a successful field experiment in which transverse planes of weakness were introduced in a concrete road slab to prevent the formation of transverse cracks elsewhere. At 40-foot intervals, reinforcing was omitted in a strip of paving 1 foot wide and a thin wooden strip 2 inches deep was inserted in the surface. This strip was later removed and the space filled with tar filler. In the 2,000-foot section constructed in this way, no transverse cracks have formed except at the planes of weakness.

C. L. McKesson, of the California Highway Commission, presented a summary of the subgrade treatment experiments at Rio Vista, Calif. His conclusions were as follows:

> That soil adulteration with cement or lime compounds is not an efficient or economical method of securing stability in heavy soils.
>
> That the suitability of soil for subgrade purposes, or the merits of various methods of soil treatment can be determined by relatively simple laboratory tests and that expensive field tests can in some cases be avoided by first resorting to a properly conducted laboratory investigation.
>
> That a sand or gravel layer is an efficient and economical method of minimizing damage to pavements resulting from swelling or shrinkage of the subsoil.

CHARACTER AND USE OF ROAD MATERIALS

R. W. Crum, of the Iowa State Highway Commission, reviewed the various investigations of the use of calcium chloride and soluble compounds of calcium chloride base as accelerators in the hardening of concrete. He finds the investigators in substantial accord that the early strength of concrete is increased and also the permanent strength when the hardening material is used in quantities less than 4 per cent of the weight of the cement. Further investigation is needed to formulate rules of practice for varying local conditions and different cements. Such admixtures are also useful to counteract to some extent the low strength resulting from curing at low temperature, but calcium chloride can not be depended upon to protect against freezing.

A review of tests of quick-hardening cements was given by F. C. Lang, of the Minnesota State Highway Department. Such cements are of two classes, high alumina and special Portland cements. Some of the high-alumina cements appear to be resistant to the action of sulphate and alkali water and all investigators agree that both classes give an early high strength. H. S. Mattimore has found that high-alumina cement concrete has compressive and transverse strengths at 24 hours greater than those of Portland cement concrete at 28 days. Mr. Lang gives information from various investigators as to time of set, early strength, quantity of mixing water, and methods of curing which will be valuable to users of these new materials.

B. A. Anderton, of the Bureau of Public Roads, reviewed the progress made during the last year in investigations of bituminous mixtures. Several investigators have brought out that with an increase in bitumen content beyond a certain point there is a rapid decrease in stability. It is the theory of some investigators that the bitumen should be sufficient, or almost sufficient, to fill the voids in the aggregate when the grains are in as close contact with each other as they are in a pavement; but it appears from other tests that maximum stability with most mixtures of the sheet-asphalt type, may be obtained with a proportion of bitumen considerably less than this amount.

Paint for highway traffic lines or zones has come into such general use that there is need for laboratory methods of determining the quality of such paints. H. S. Mattimore, of the Pennsylvania State Highway Department, has made a helpful contribution in this direction through a discussion of physical tests for consistency, spreading rate, hiding power, drying time, light resistance, visibility, and durability.

HIGHWAY TRAFFIC ANALYSIS

The committee on highway traffic analysis presented discussions of the 3-lane, 2-way roadway, traffic analysis, elimination of obstacles impeding the free flow of traffic, and adequate rights of way for future highway development.

The committee concludes that 22 and 24-foot roadways are not needed for two lanes of traffic. Roadways 24 to 30 feet in width are used as 3-lane, 2-way roadways. Due to the impossibility of governing traffic on the middle lane by right-of-way regulations the committee recommends 4-lane roadways where a 2-lane roadway is not adequate.

The discussions of the committee were supplemented by papers by J. G. McKay, of the Bureau of Public Roads, dealing with population and highway traffic and interstate traffic on Federal-aid highways. The former paper discusses population and population trends in relation to present and future highway traffic and gives a brief résumé of the analysis of this phase of the problem in the Cook County transportation survey.

In the paper on interstate traffic on Federal-aid highways Doctor McKay presented figures from various traffic surveys showing the large percentage of interstate travel and pointed out that highways are no longer neighborhood affairs to be discussed in town meeting and to be maintained by local authorities.

HIGHWAY FINANCE

The committee on highway finance substituted for its report the report of the special investigation of urban aspects of the highway finance problem conducted under the auspices of the Highway Research Board by Jacob Viner, of the University of Chicago. This study had for its purpose an examination of those problems connected with the financing of highways which affect most closely the governments and residents of urban communities.

The study deals with three phases of the problem:

1. The special interests of cities and of urban motor-vehicle owners in the highway finance policies and practices of State and county governments.
2. The methods and problems of urban highway finance.
3. The financial aspects of the traffic congestion problem.

HIGHWAY MAINTENANCE

Seven different maintenance problems were studied during the year by subcommittees to which they were assigned and the subcommittee reports were used in formulating the report of the main committee. This report discussed dust prevention and surface treatment of gravel and macadam roads, crack fillers for concrete pavements, snow removal and snow-removal equipment, highway signs, standardized maintenance accounting, relation between age of pavement and its annual maintenance cost, coverings for poorly constructed and disintegrating concrete roads, and a description of an instrument for measuring road roughness.

In discussing the surface treatment of gravel roads it is stated that a season's application of calcium chloride varies in cost from $250 to $300 per mile, as against $300 to $500 per mile for light asphaltic oil. The opinion is very general that the minimum traffic for which light asphaltic oil or calcium chloride treatment is justified ranges from 300 to 500 vehicles per day, and the minimum traffic justifying a bituminous or mat-forming surface treatment ranges from 500 to 800 vehicles per day.

A survey by the committee indicates that there is considerable variation in practice as to crack fillers for concrete pavements. Some States use asphalt in its various forms, while others use tar. Some require a penetration as low as 35, whereas others require up to 250. Most of the States use coarse or fine sand as covering material, but a few have experimented with other materials. Experiments are now under way in Illinois, Connecticut, Michigan, and California with various combinations of material.

The review of snow-removal activities shows that an increasing mileage is being kept open each year and that improved methods and machinery are being used, but there has been no unusual development in this field.

In discussing maintenance accounting the committee suggests that such accounting should cover quantities of work performed and unit costs as well as purposes, totals, and places of expenditures. It recommends that the standard system of accounting as adopted by the American Association of State Highway Officials be put into operation by all of the States as soon as possible.

The committee feels that the question of type of resurfacing for old and worn concrete roads is of increasing importance. It is not yet ready to offer conclusions, and lists a number of projects which have been resurfaced and which should be studied to determine the best methods.

PROGRESS MADE IN SPECIAL INVESTIGATIONS

S. S. Steinberg, of the Highway Research Board, presented a report on the work that has been done in getting under way the special investigation of the board on the development of earth roads. The objects of the investigation are to coordinate the data already available on the improvement of earth roads and to stimulate further research in order to find an inexpensive road surface to carry intermediate traffic. No field investigations have yet been made, but the material assembled will enable the investigator to study effectively in the field the experiments completed or under way in many sections of the country.

R. W. Crum and Mark Morris, of the Iowa State Highway Commission, presented a progress report on the culvert investigation undertaken by the Highway Research Board. Fourteen States have been visited and 516 installations inspected. The report discusses characteristic types installed and defects observed, with causes.

REINFORCEMENT IN CONCRETE ROADS

Part II of the proceedings contains the report of the investigation of the economic value of reinforcement in concrete roads by C. A. Hogentogler. In this survey a general inspection was made of 5,500 miles of road in a search for plain concrete surfaces which had disintegrated, reinforced surfaces which had failed, and cracks in reinforced concrete pavements of such width as to indicate broken steel. Detailed inspections were made of about 2,000 miles of concrete road in 26 States, including thicknesses of 5 to 10½ inches, ages from 1 to 13 years, subgrades ranging from stable sands to bad clays, all conditions of traffic, and reinforcement ranging from light wire fencing to ⅞-inch-bar mats.

The report includes sketches showing detailed conditions as regards cracks and breakage, photographs, design details, road condition charts, digests of pertinent highway researches and reports of allied investigations furnished by cooperating agencies. In a summary of conclusions 14 points are presented among which are the following:

> The amount of cracking and subsequent disintegrating is a function of time; thus, the rate of cracking is a measure of the life of the pavement.
>
> The data show that steel reinforcement reduced the rate of cracking and thus increased the life of the pavement. This applies both to concrete pavements and other pavements laid upon a concrete base.
>
> Crack reduction is more economically accomplished by the use of steel reinforcement than by additional thickness of concrete.
>
> A greater reduction was afforded by small steel members closely spaced than by larger members widely spaced.

Copies of the proceedings may be obtained by addressing the Highway Research Board, National Research Council, Washington, D. C.

(Continued from p. 124)

China.—M. Ouang Hang, engineer of the railways, Paris representative of the Ministry of Communications, Peking.

United States.—(In collaboration): Messrs. Thomas H. MacDonald, Chief, U. S. Bureau of Public Roads, Washington, D. C.; John A. Macdonald, State highway commissioner, Hartford, Conn.; W. G. Sloan, State highway engineer, Trenton, N. J.

France.—(In collaboration): Messrs. E. Chaix, vice president of the A. C. F., president of the Tourist and General Traffic Commission of the A. C. F., director of the National Touring Office; Gaston Raffard, member of the Touring and General Traffic Commission of the A. C. F., 8 Place de la Concorde, Paris.

Great Britain.—Sir Lynden Macassey, K. B. E., M. A., Ll. D., etc., 24 Sloane Gardens, London S. W. 1.

Italy.—(In collaboration): Messrs. Avv. Francesco La Farina, Ministry of Public Works, Rome; Eng. Gaetano D'Alò, director of the inspection department of railways, tramways, and motors, Milan; Marquis Avv. Paolo Sommi-Picenardi, director of the Automobile Club, Milan; Alberto Depetrini, section chief of the Ministry of Public Works, Rome.

Low countries.—(In collaboration): Messrs. D. A. Van Heyst, chief engineer of the "Rykswaterstaat" at The Hague, 121 Bezuidenhout; G. J. Van den Broek, chief engineer of the "Rykswaterstaat," 1 Surinamestraat, The Hague; P. J. Van Woorst Vader, jr., chief engineer of the "Rykswaterstaat," 13 Florapark, Haarlem.

Sweden.—Mr. Adler (Knut Kristian), captain of the Bridges and roads Corps, Linkoeping.

GENERAL SPEAKERS

First question: Engineer Angelo Rampazzi, section president at the Board of Public Works, Rome.

Second question: Engineer Italo Vandone of the Italian Touring Club, Milan.

Third question: Engineer Michelangelo Böhm, 45 Via De Amicis, Milan.

Fourth question: Engineer Luigi Frosali, chief engineer of the Technical Bureau of the Province of Florence.

Fifth question: Engineer Cesare Albertini, of the Technical Bureau of the Milan Municipality.

Sixth question: Grand Uff. Michele Carlo Isacco, managing director of traffic and ports, Ministry of Public Works, Rome.

VIRGINIA BUILDING DEMONSTRATION ROAD

HISTORIC BULL RUN BATTLEFIELD THE SITE OF IMPORTANT HIGHWAY INVESTIGATIONS

Reported by C. A. Hogentogler, Highway Engineer, U. S. Bureau of Public Roads

A DEMONSTRATION highway from which it is expected to obtain information comparable in value and importance with that produced by the Bates Road and Pittsburg (Calif.) tests and the experiments of the Bureau of Public Roads at Arlington, Va., is now under construction between Fairfax and Warrenton, Va. The road, which is being built by the Virginia Highway Commission under the directon of Henry G. Shirley, chairman, and C. S. Mullen, chief engineer, of the commission, is a Federal-aid project. It traverses the historic Bull Run battlefield and the famous bridge over the run still may be seen by visitors to the demonstration road.

The new demonstration, unlike the previous experimental roads, is not intended as a test of road design or of the relative value of different surfacing materials. The Virginia standard 8–6–8 concrete section, which is used on the major portion of the road, is not in question. Instead the type of information sought concerns a number of questions not covered by the previous tests and relating principally to the influence of the subgrade, the effect of admixtures of various sorts, of the kind and quantity of reinforcement, of various finishing methods, and other details of construction. In general, the type of information sought concerns the economy of various common practices in concrete construction, especially the various measures employed to prevent cracking and breaking and promote the smoothness of the surface.

The Famous Stone Bridge Crossing Bull Run

In its length of 9 miles the road provides opportunity for the study of the effect of the following variables:

1. *The subgrade.*—Seven different types of soil are represented in the natural subgrade and thin courses of stone screenings and both single and double layers of tar paper have been used as coverings in certain sections.

2. *Construction joints and dividing planes.*—Expansion and center joints are used in only one section. Construction joints are placed at noon and at night. Longitudinal dividing planes have been used in two sections, in one of which transverse dividing planes have also been used. These planes of weakness are formed by installation of a patented type of separating strip about 2½ inches deep. With the exception of the section containing both longitudinal and transverse dividing planes, which is laid to a uniform thickness of 7 inches, the Virginia design of 8–6–8 pavement, 18 feet wide, was used in all sections.

3. *Aggregate, cement, and mix.*—The same sand and crushed stone are used throughout the work. Two well-known brands of cement have been used, and the mix is either 1 : 2½ : 3½ or 1 : 2 : 4.

4. *Admixtures.*—The three admixtures used are hydrated lime, 5 and 8 per cent; Celite, 3 per cent; and calcium chloride, 2 per cent; all incorporated.

5. *Types of construction.*—The three types of construction employed are plain concrete, vibrolithic concrete, and reinforced concrete. Both single and double layers of welded steel fabric, bar mats, and expanded metal are used in the latter.

6. *Finishing and curing.*—Part of the road is being finished by hand methods. The remainder is to be machine finished, using both Ord and Lakewood finishing machines. With the exception of the section containing calcium chloride, all sections are to be cured alike by covering with wet earth for 14 days.

EXPERIMENTAL WORK

The road on which the demonstration is being conducted is strictly a service highway constructed under State supervision with convict labor. It will not be subjected to accelerated traffic.

The Bureau of Public Roads is cooperating in the experimental work which includes studies of the subgrade, the concrete materials, and the effect of construction operations. The nature of the investigations is briefly as follows:

1. *Material investigation.*—This work, which is carried on at the Arlington Experiment Farm, includes tests of specimens in which either the sand, cement, or stone is varied, and tests, at various ages, of beams made of the same mixes and containing the same admixtures as the road sections. Routine tests and special expansion and contraction observations are being made in the Washington laboratories of the bureau.

2. *Test cylinders.*—Test cylinders made during construction are being cured alongside of the road under conditions as nearly as possible identical with the road slab. The position of batches from which the cylinders are made is marked so that cores can be drilled later. This will allow a definite comparison to be made between the strengths of cores and cylinders from the same batch.

3. *Surveys and construction observations.*—A complete subgrade survey was made prior to the beginning of construction in the course of which observations of drainage conditions were taken and samples of soil were procured for testing in the laboratory. Subsequent inspections of soil conditions have been made after heavy rains.

Observers stationed on the work are keeping accurate records of all occurrences and practices which might influence the condition of the pavement. Atmospheric conditions, temperature and humidity, and the slump of the concrete are recorded hourly. The time of pouring each batch and the time it is finished and covered is kept accurately and the position of the batch is carefully platted with identifying station numbers indicated. This provides an efficiency record which shows the variation in the intervals between the three important stages.

Condition surveys of the surface are made before the wet earth covering is placed and after it is removed; and all of the data mentioned are plotted on final record sheets together with the grade and alignment of the road. Future surveys will be made to determine how the surface condition is influenced by the various factors.

4. *Observations of surface smoothness.*—After the earth covering is removed the relative smoothness of the various sections will be determined by the use of a 16-wheel profilometer. The effect of the surface condition on truck wheel impact will be measured by means of accelerometers attached to trucks; and the effect on the truck body will be measured with a recording roughometer.

Information with regard to the surface smoothness and relative first costs of the various sections will be available immediately. The influence of the various other factors on the condition of the surface can, naturally, be learned only after the road has been in service a sufficient length of time to develop defects.

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
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1279D. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

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331C. Standard Specifications for Corrugated Metal Pipe Culverts.

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Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
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Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JULY 31, 1926

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,725,985.09	1,298.3	$ 1,033,288.73	$ 492,916.35	75.7	$ 4,036,643.28	$ 1,865,385.81	131.4	$ 208,862.54	$ 104,431.27	11.5	$ 3,160,736.48	Alabama
Arizona	10,949,878.25	5,863,772.35	729.8				1,401,980.83	945,372.27	81.4	339,708.89	236,515.53	26.3	2,571,588.85	Arizona
Arkansas	18,364,544.50	7,666,698.35	1,323.0	59,066.03	29,533.01	5.1	4,664,865.94	2,216,116.50	303.7	706,048.44	348,326.76	54.9	1,345,127.38	Arkansas
California	27,142,596.90	13,003,592.30	1,058.0				11,873,095.16	5,790,115.12	320.3	191,797.75	110,673.41	0.5	3,168,434.17	California
Colorado	13,905,904.64	7,127,268.18	745.0	439,200.74	236,115.35	8.2	4,044,756.65	1,972,893.38	207.5	769,494.53	361,022.34	35.1	2,628,492.76	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1				3,867,110.97	1,019,568.03	54.6	862,699.27	327,521.25	8.8	886,005.92	Connecticut
Delaware	4,918,052.29	1,761,665.60	124.3				1,115,536.81	470,577.90	28.2	623,535.35	218,310.90	15.7	3,503.60	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	420,473.69	208,262.41	20.9	9,368,343.27	4,362,418.06	268.9	241,489.11	119,798.28		1,570,112.96	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	551,256.85	274,533.75	30.8	11,872,496.51	5,829,003.86	630.0	1,240,213.28	557,285.51	24.6	106,892.02	Georgia
Idaho	11,061,198.14	5,882,112.70	724.7	254,585.56	130,664.38	6.9	2,737,287.60	1,663,337.23	155.8	1,056,756.72	616,474.67	90.3	267,038.02	Idaho
Illinois	44,116,611.86	20,619,995.74	1,377.7	2,012,307.84	980,960.66	68.1	5,230,338.91	2,508,807.62	180.6	814,171.32	378,771.83	25.8	5,343,562.16	Illinois
Indiana	16,949,425.87	8,172,125.19	534.3	603,411.29	295,697.26	18.4	18,501,800.32	8,713,519.50	528.4	281,117.68	140,558.84	7.2	882,454.21	Indiana
Iowa	29,062,375.40	11,926,302.10	2,114.8	132,077.35	66,038.67	9.9	11,114,136.87	5,351,462.04	705.6	3,160,525.11	1,442,228.21	142.4	699,531.98	Iowa
Kansas	32,826,601.64	12,590,489.25	1,160.6	455,951.75	202,404.07	39.6	11,901,412.35	4,644,543.66	661.4	1,833,784.87	670,215.00	132.9	1,356,759.12	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	307,209.39	106,386.83	6.3	5,556,069.73	2,669,005.95	293.6	95,483.33	47,741.66	3.6	1,897,592.31	Kentucky
Louisiana	13,830,692.68	6,144,739.99	1,065.9				3,448,051.45	1,672,702.16	161.2	1,118,622.37	605,003.13	37.6	949,962.72	Louisiana
Maine	8,747,562.76	4,192,507.39	303.6				2,354,417.18	864,849.24	67.3	1,446,937.89	607,067.98	50.0	810,403.39	Maine
Maryland	10,924,943.10	5,112,991.22	423.3				513,398.28	262,643.43	30.8	1,287,472.78	538,920.86	57.6	20,501.49	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.6				4,557,049.89	1,229,562.32	61.1	944,487.76	247,578.55	14.5	1,974,224.61	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	215,575.91	103,606.96	7.6	11,169,960.39	5,102,817.17	295.2	173,599.00	81,626.50	11.2	3,227,262.07	Michigan
Minnesota	37,170,985.95	15,586,116.56	3,181.9	663,725.36	300,359.07	66.9	9,100,131.01	3,431,500.00	524.3	443,985.90	31,000.00	11.1	242,804.37	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1,129.0	82,987.42	40,629.00	6.9	6,988,798.36	3,463,229.63	380.3	1,705,272.78	784,437.54	80.0	425,187.73	Mississippi
Missouri	28,989,166.92	13,736,014.85	1,543.2	821,068.97	383,099.04	25.8	19,820,709.91	7,800,913.06	541.7	1,056,788.43	465,838.49	19.3	400,570.56	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	94,415.49	60,088.27	2.0	1,663,293.14	1,138,296.07	130.8	1,784,353.74	1,353,224.13	149.6	4,539,810.64	Montana
Nebraska	11,533,401.62	5,474,202.52	1,768.3	96,921.54	48,218.16	15.3	13,380,938.73	6,558,392.03	1,366.0	1,279,000.83	613,900.31	126.4	1,940,521.98	Nebraska
Nevada	7,558,195.51	5,130,934.59	538.8	660,130.91	574,971.17	65.1	2,785,464.77	2,392,400.87	323.8	13,387.17	11,743.22		585,155.15	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6				948,617.15	437,535.18	28.7	405,722.18	182,933.16	10.0	171,573.59	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	156,206.43	63,486.76	4.0	6,823,899.73	2,503,716.67	30.8	2,028,186.61	431,160.00	28.7	370,715.36	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0				1,457,040.48	970,998.88	100.4	936,041.99	661,614.90	65.2	2,000,114.84	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	1,275,861.14	487,774.68	28.9	31,802,920.00	8,721,902.70	553.9	9,965,403.00	2,181,550.00	133.4	4,742,010.43	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9				8,035,569.06	3,484,782.13	190.1	635,619.50	317,809.75	26.1	737,276.18	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	33,059.36	16,529.68		6,147,730.12	3,151,787.32	888.1	2,781,831.35	1,602,393.13	277.9	46,089.09	North Dakota
Ohio	47,699,532.90	17,371,787.03	1,364.1	558,568.83	257,865.98	25.5	11,295,034.17	4,348,790.47	336.9	3,887,394.67	1,427,033.34	111.4	2,326,319.18	Ohio
Oklahoma	28,247,950.33	13,159,999.16	1,178.9	38,958.17	19,233.78	5.2	2,852,788.00	1,334,877.42	92.2	877,244.37	418,679.81	71.2	1,127,096.84	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	112,462.11	60,931.58	5.9	3,191,801.62	1,750,354.69	120.4	409,330.64	237,616.61	22.1	237,229.33	Oregon
Pennsylvania	51,366,150.80	21,560,732.04	1,188.8				25,913,565.91	7,326,756.54	629.6	2,919,635.07	933,609.06	68.4	1,618,783.36	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7				1,531,802.80	427,155.00	28.6	485,392.16	124,365.00	8.3	557,219.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1,481.9	388,144.47	152,702.67	34.7	5,856,691.37	2,754,874.76	205.6	362,029.99	114,921.58	19.3	12,702.06	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,161.2	261,269.44	130,634.71	67.0	3,585,246.43	1,806,308.75	603.7	501,057.09	269,552.46	99.7	356,467.11	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0				7,850,214.11	3,673,669.05	253.9	1,198,242.23	518,210.12	19.0	912,127.81	Tennessee
Texas	69,183,673.48	27,440,254.72	4,920.2	345,850.83	153,965.47	47.9	18,339,531.39	8,100,493.13	859.6	2,959,408.68	1,334,962.53	65.7	3,576,755.15	Texas
Utah	8,253,178.03	5,098,440.68	546.4				1,707,708.64	1,291,288.39	154.0	848,405.31	646,544.22	56.8	782,505.71	Utah
Vermont	4,242,042.64	2,017,699.51	134.5				1,941,881.25	764,535.39	36.1	161,450.52	41,736.47	2.9	444,535.63	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.6	373,701.36	166,360.34	10.2	5,062,610.04	2,647,371.87	175.0	664,784.79	245,118.85	18.8	56,934.83	Virginia
Washington	17,078,511.63	7,782,909.46	668.6				3,309,677.73	1,653,600.00	41.0	1,250,203.66	453,000.00	50.4	256,266.64	Washington
West Virginia	9,473,716.44	4,141,062.66	392.9	457,126.16	193,696.41	12.4	4,805,344.41	1,869,689.67	124.7	1,665,915.01	754,239.62	62.8	393,822.65	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1				7,892,348.60	3,796,814.62	357.9	814,717.03	393,743.00	57.8	2,855,551.65	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.6	341,141.32	218,169.00	48.0	2,481,191.35	1,564,084.02	189.4	215,935.78	150,342.81	48.7	592,771.12	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	13,246,004.43	6,455,854.47	769.2	347,941,199.49	148,512,656.64	14,329.0	59,652,449.37	24,261,152.56	2,462.4	65,966,632.75	TOTALS

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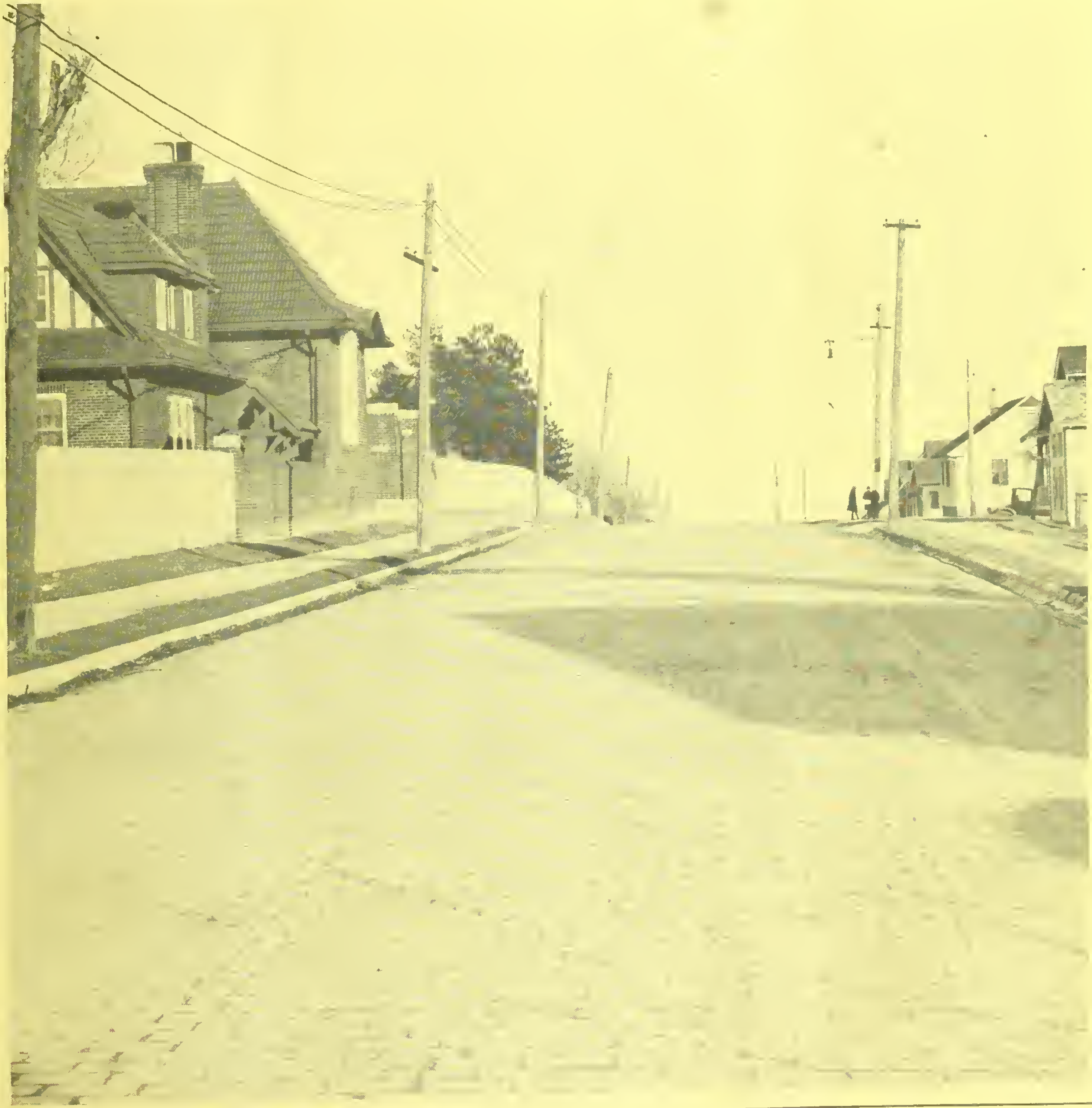
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 7 ▽ SEPTEMBER, 1926



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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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VOL. 7, NO. 7 SEPTEMBER, 1926 H. S. FAIRBANK, Editor

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THIN BRICK PAVEMENTS STUDIED

REPORT OF ACCELERATED TRAFFIC TESTS AND FIELD STUDIES BY THE BUREAU OF PUBLIC ROADS

Reported by L. W. TELLER, Engineer of Tests, and J. T. PAULS, Associate Highway Engineer, United States Bureau of Public Roads

THERE has been manifest lately a growing belief among engineers that brick less than 4 inches in thickness may properly be used in the construction of brick pavements. Advocates of the thinner brick have contended that the 4-inch depth is unnecessary, and that brick of 3-inch thickness or even less would give equally satisfactory results, and the extensive and satisfactory use of 3½ and even 3 inch brick in some parts of the country has apparently lent support to the contention.

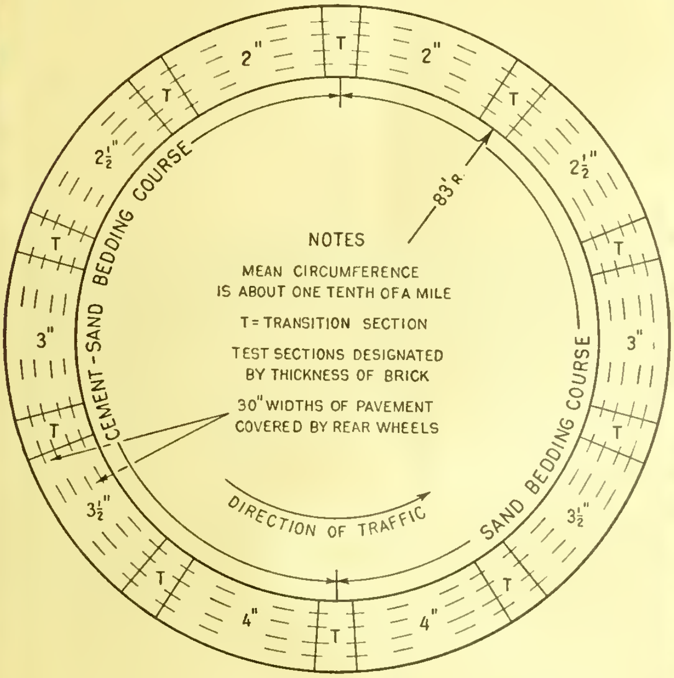

2"
T
2"
T
2½"
T
2½"
T
BEDDING COURSE
83' R.
NOTES
MEAN CIRCUMFERENCE IS ABOUT ONE TENTH OF A MILE
T = TRANSITION SECTION
TEST SECTIONS DESIGNATED BY THICKNESS OF BRICK
30" WIDTHS OF PAVEMENT COVERED BY REAR WHEELS
3"
CEMENT-SAND
3"
SAND BEDDING COURSE
T
T
3½"
DIRECTION OF TRAFFIC
3½"
T
T
4"
T
4"

FIG. 1.—GENERAL PLAN OF BRICK TEST TRACK

If such a reduction in thickness can be made without impairing the service value of the pavement, and if the manufacture of the thinner brick is practicable, the resulting economy in the construction of brick surfaces would doubtless be very considerable; and the question is one which merits thorough investigation on that account. If, as experience has apparently demonstrated, the 3½ and 3 inch thickness can be employed successfully, the investigation should confirm that fact and go further to the extent of ascertaining the least thickness practicable from the points of view of manufacture, service, and cost.

Recognizing the importance of the question the Bureau of Public Roads has undertaken to conduct such an investigation, in the course of which it has made a field study of the service behavior of brick pavements in which brick of less than 4-inch thickness have been used, and concurrently an accelerated traffic test, at Arlington, Va., on sections of pavement built of brick of different thickness and a series of laboratory tests on the brick used to determine their quality. Several plants manufacturing brick of less than 3-inch thickness have also been visited to determine the attitude of the industry toward the use of the thinner brick and to ascertain if their manufacture presents any particular difficulties.

CONCLUSIONS INDICATED BY THE INVESTIGATION

The several parts of the investigation have now been completed and the analysis of the data obtained seem to warrant certain conclusions, among which the more important are:

1. That 2½-inch brick of the quality used in the Arlington traffic tests, when properly supported, will prove satisfactory for pavements carrying the heavier types of traffic.
2. That brick of 2-inch thickness, when properly supported and of the quality used in the tests, will be adequate for pavements on streets carrying the lighter types of traffic.
3. That a bedding course of plain sand is more effective in reducing breakage of brick than a cement-sand bedding course, the breakage being much less on the former than the latter. The depth of the sand bedding course should not greatly exceed three-fourths inch. Increasing the depth tends to produce roughness in the pavement.
4. That cobbling of the brick is greatly increased as the spacing between bricks is increased.
5. That the use of excessive quantities of asphalt filler is a common and serious fault in construction, unnecessarily increasing the cost and resulting in a condition which impairs both the appearance and the serviceability of the pavement.
6. That base construction of other than the rigid type may in many cases prove entirely satisfactory.

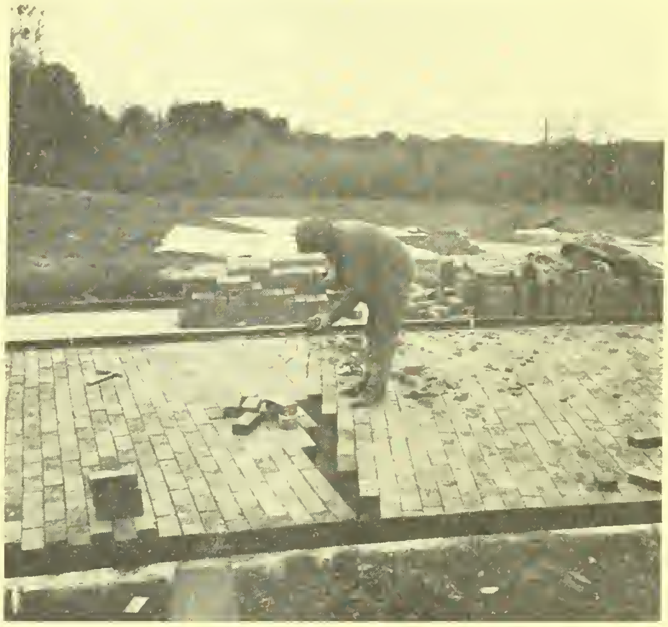


FIG. 2.—BRICKLAYER CLOSING IN A "DUTCHMAN." THIS FEATURE WAS NECESSARY TO COMPENSATE FOR THE CURVATURE OF THE TEST TRACK

Macadam bases and those constructed of certain types of natural earth appear to be suitable when the local conditions are such that these types of construction maintain their stability throughout the year.

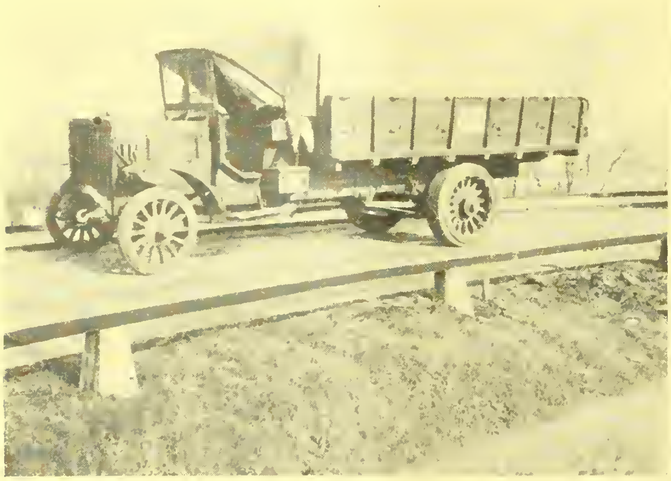


Fig. 3.—Type of Truck and Solid Rubber Tires Used During the First Phase of the Test

7. That no difference in the base construction is necessary for the different thicknesses of brick.

THE ACCELERATED TRAFFIC TESTS

The accelerated traffic tests have been carried on at the Arlington Experiment Farm, Arlington, Va., during the last several months. Their object was to obtain data on the relative resistance to heavy-truck traffic of paving brick of the several thicknesses, and every effort was made, therefore, to eliminate all other variable factors which might influence the results of the tests, the only exception being the use of the two kinds of bedding course. Because of the opportunity afforded by the test to study the relative merits of the plain sand and cement-sand bedding, it was decided to include this feature, and accordingly the pavement as laid includes duplicate sections of each thickness of brick, one on each of the two types of bedding.

In order to minimize the possibility of a variation in the quality of the brick used, they were all obtained from one manufacturer, and all are of the vertical-fiber, plain wire-cut, lugless type, 8½ inches long and 4 inches wide, the depths for the several sections being 2, 2½, 3, 3½, and 4 inches.

A circular concrete base, which formerly had served in tests of bituminous pavements, was available and was used as a base for the brick sections. This base has a mean circumference of about 540 feet, is 13 feet wide, and at the beginning of these tests was in perfect condition.

For the purpose of the brick test this circular base was divided symmetrically into 10 equal sections. On one-half of the circle the plain sand bedding course was laid to a thickness of three-fourths inch; on the other half a 1:4 cement-sand course of the same thickness was used. On each type of bedding five test sections were constructed, one of each thickness of paving brick. Each section was about 45 feet long and between them the change of one-half inch in surface elevation made necessary by the change in thickness of brick was made in a transition section about 10 feet long.

This change in elevation was accomplished by adjusting the thickness of the bedding course over a length of about 3 feet in the center of the transition section, the bedding for this distance being stiffened by the addition of a small quantity of Portland cement. The general plan of the test track and the relative position of the various sections are shown in Figure 1.

On account of the difference in circumference between the inside and outside edges of the track, it was necessary to give special attention to the manner in which the brick were laid around the circle; and it was decided that the best method would be to lay the brick in a series of short tangents and to join these tangents with "Dutchmen." One of these is shown in process of construction in Figure 2. In the track when completed there was one "Dutchman" in the center of each test section and another in each transition section; and in this way a uniform width of joints was maintained throughout the entire pavement.

After the brick were laid the pavement was rolled with a 3-ton tandem roller, and bricks which appeared to be damp were dried with a portable kerosene torch. This was followed by culling, after which the joints were filled with a squeegee coat of asphalt of 32 penetration, applied at a temperature of 375° to 400° F. The pavement was constructed late in November and on account of the low temperature considerably more asphalt adhered to the surface of the pavement than was desirable. Better filling of the joints would have resulted if higher air temperatures had prevailed. A light coat of sand followed the asphalt and another rolling completed the construction of the test pavement. All work was done by a contractor thoroughly familiar with brick pavement construction.

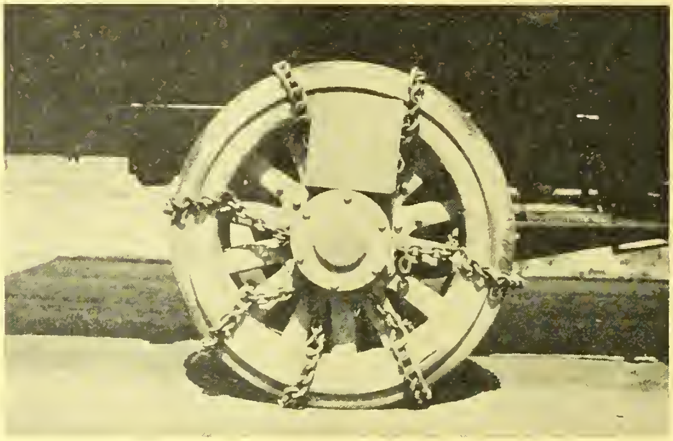


Fig. 4.—Truck Wheel Equipped with Nonskid Chains During the Second Phase of the Traffic Test

Before traffic was applied the various sections were marked off with radial painted lines, and longitudinal traffic lines were also painted on the surface in order that the truck wheels might be confined to a path 30 inches wide and thus accelerate the test.

The general plan was to apply 3-ton, 5-ton, and 7½-ton motor-truck loads a definite number of times on each test section and to observe the results. The first phase consisted of the application of these loads with motor trucks equipped with solid rubber tires in good condition, as shown in Figure 3. In the second phase the trucks were equipped with heavy nonskid chains on the rear wheels, as shown in Figure 4. The details of the loading program for the two phases of the test are given in Table 1.

Table 1.—*Loading program for the two phases of the accelerated traffic test*

FIRST PHASE

Rated load	Maximum wheel load	Tire size	Load per inch of tire width	Number of trips over each test section
Tons	*Pounds*	*Inches*	*Pounds*	
3	5, 800	40 by 10	580	10, 000
5	7, 750	40 by 12	646	10, 000
7½	10, 570	40 by 12	881	20, 000

SECOND PHASE

Rated load	Maximum wheel load	Tire size	Load per inch of tire width	Chains			Number of trips over each test section
				Diameter	Height above tire	Number per wheel	
Tons	*Pounds*	*Inches*	*Pounds*	*Inch*	*Inches*		
3	5, 800	40 by 10	580	½	1⅛	7	10, 000
5	7, 750	40 by 12	646	½	1⅛	8	10, 000
7½	10, 570	40 by 12	881	½	1⅛	8	2, 200

The base of the test track was originally constructed with superelevation for a speed of 9 miles per hour. It was thought that this speed was too low for representative traffic, and during the tests without chains a speed of 12 miles per hour was maintained. This caused a difference in pressure under the two rear wheels, the effect of which will be discussed later. During the second phase of the test it was found that the trucks could not maintain a speed of over 9 miles per hour without overheating because of the heavy chains, so this speed was used throughout this part of the traffic test.

RESULTS OF THE ACCELERATED TRAFFIC TEST

The results of the accelerated traffic test as measured by the percentage of the total number of brick in the two 30-inch wheel strips of each section, broken transversely under each load in the two phases of the test, are shown in Table 2. The same data expressed in numbers of broken brick, graphically for the 2-inch brick and in tabular form for the other thicknesses, are shown in Figure 5. These are the data from the traffic test. The results of the physical tests of the brick and the compression tests made on brick taken from the pavement after the completion of the traffic tests are shown in Tables 3 and 4, respectively.

Table 2.—*Percentage of total number of brick in the two wheel strips of each section, broken transversely under each load*

Tire condition	Number of trips	Rated load	Thickness of brick									
			2-inch		2½-inch		3-inch		3½-inch		4-inch	
			Sand bed	Cement-sand bed	Sand bed	Cement-sand bed	Sand bed	Cement-sand bed	Sand bed	Cement-sand bed	Sand bed	Cement-sand bed
Plain solid tires		*Tons*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*
	10, 000	3	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0
	10, 000	5	1. 6	2. 1	. 0	. 0	. 0	. 0	. 0	. 0	. 0	. 0
	10, 000	7½	5. 6	11. 8	. 2	. 3	. 0	. 0	. 0	. 4	. 0	. 2
	10, 000	7½	3. 5	4. 6	. 0	. 0	. 0	. 0	. 0	. 0	. 0	. 0
Total	--------	------	10. 7	18. 5	. 2	. 3	. 0	. 0	. 0	. 4	. 0	. 2
With nonskid chains	10, 000	3	1. 0	6. 0	. 0	. 5	. 3	. 0	. 0	. 1	. 0	. 0
	10, 000	5	. 5	3. 1	. 0	. 5	. 0	. 0	. 1	. 0	. 0	. 1
	2, 200	7½	. 4	1. 1	. 0	. 0	. 0	. 0	. 0	. 0	. 0	. 0
Total	--------	------	1. 9	10. 2	. 0	1. 0	. 3	. 0	. 1	. 1	. 0	. 1

Table 3.—*Results of physical tests on the brick used in the test pavement at Arlington Experiment Farm, Va.*

Brick thickness	Rattler loss by weight	Modulus of rupture		Crushing strength on edge
		Tested flat	Tested on edge	
Inches	*Per cent*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
2	22. 8	2, 088	1, 991	10, 240
2½	18. 8	2, 461	2, 197	12, 530
3	19. 0	2, 115	1, 964	10, 770
3½	17. 0	2, 233	2, 146	10, 916
4	16. 8	2, 117	1, 998	10, 850

Table 4.—*Results of compression tests on the different sizes of brick in various conditions taken from the test pavement after the completion of all traffic*

Condition of cobbling	Brick thickness	Average crushing strength on edge (pounds per square inch)						
		Not broken transversely	Traffic stage in which transverse break occurred[1]					
			A	B	C	D	E	F
	Inches							
Minimum	2	8, 370	--------	6, 090	--------	10, 980	9, 170	9, 240
Average	2	8, 490	7, 550	6, 260	8, 290	6, 740	--------	9, 570
Maximum	2	7, 490	7, 520	6, 220	7, 920	5, 770	--------	------
Minimum	2½	9, 840	--------	--------	--------	--------	--------	------
Average	2½	9, 240	--------	9, 970	9, 340	10, 810	11. 010	------
Maximum	2½	8, 300	--------	--------	7, 270	9, 790	--------	------
Minimum	3	10, 110	--------	--------	--------	--------	--------	------
Average	3	8, 540	--------	--------	--------	--------	--------	------
Maximum	3	8, 180	--------	--------	--------	--------	--------	------
Minimum	3½	9, 860	--------	--------	--------	--------	--------	------
Average	3½	9, 810	--------	--------	8, 150	--------	9, 940	------
Maximum	3½	6, 200	--------	--------	--------	10, 120	--------	------
Minimum	4	11, 090	--------	--------	--------	--------	--------	------
Average	4	9, 980	--------	--------	9, 530	--------	8, 630	------
Maximum	4	10, 730	--------	--------	--------	--------	--------	------

[1] Traffic stages.

A. Includes 10,000 trips with 3-ton load and plain solid tires (in which no breaks occurred) and 10,000 trips with 5-ton load and plain solid tires. Breaks under this traffic are indicated in the photographs by a solid circle.

B. Includes A plus 10,000 trips with 7½-ton load and plain solid tires. Breaks under this traffic are indicated in the photographs by an open circle.

C. Includes A and B plus 10,000 trips with 7½-ton load and plain solid tires Breaks under this traffic are indicated in the photographs by a triangle.

D. Includes A, B, and C plus 10,000 trips with 3-ton load and plain solid tires with nonskid chains. Breaks under this traffic are indicated in the photographs by a single line.

E. Includes A, B, C, and D plus 10,000 trips with 5-ton load and plain solid tires with nonskid chains. Breaks under this traffic are indicated in the photographs by a cross.

F. Includes A, B, C, D, and E plus 2,200 trips with 7½-ton load and plain solid tires with nonskid chains. Breaks under this traffic are indicated in the photographs by an open square.

From a study of the graphs in Figure 5 it will be seen that:

1. Practically all transverse breakage occurred in the 2-inch brick sections.
2. Within the limits of the test, resistance to breakage by the 2½-inch brick appears to have been but slightly less than that of the thicker brick.
3. Breakage in sections laid on sand bedding is less than half of that occurring in sections laid on cement-sand bedding.
4. The greatest amount of breakage occurred during the application of the 7½-ton load with plain solid tires.
5. The greatest increase in breakage occurred during the first 10,000 trips of the 7½-ton, plain-solid-tired traffic.
6. The rate of breakage greatly decreased under the traffic following completion of the 7½-ton, plain-solid-tired traffic.

The high resistance to breakage shown by the 2½-inch brick was one of the important results obtained from this test. The slightly better quality indicated by the physical tests can only partially explain

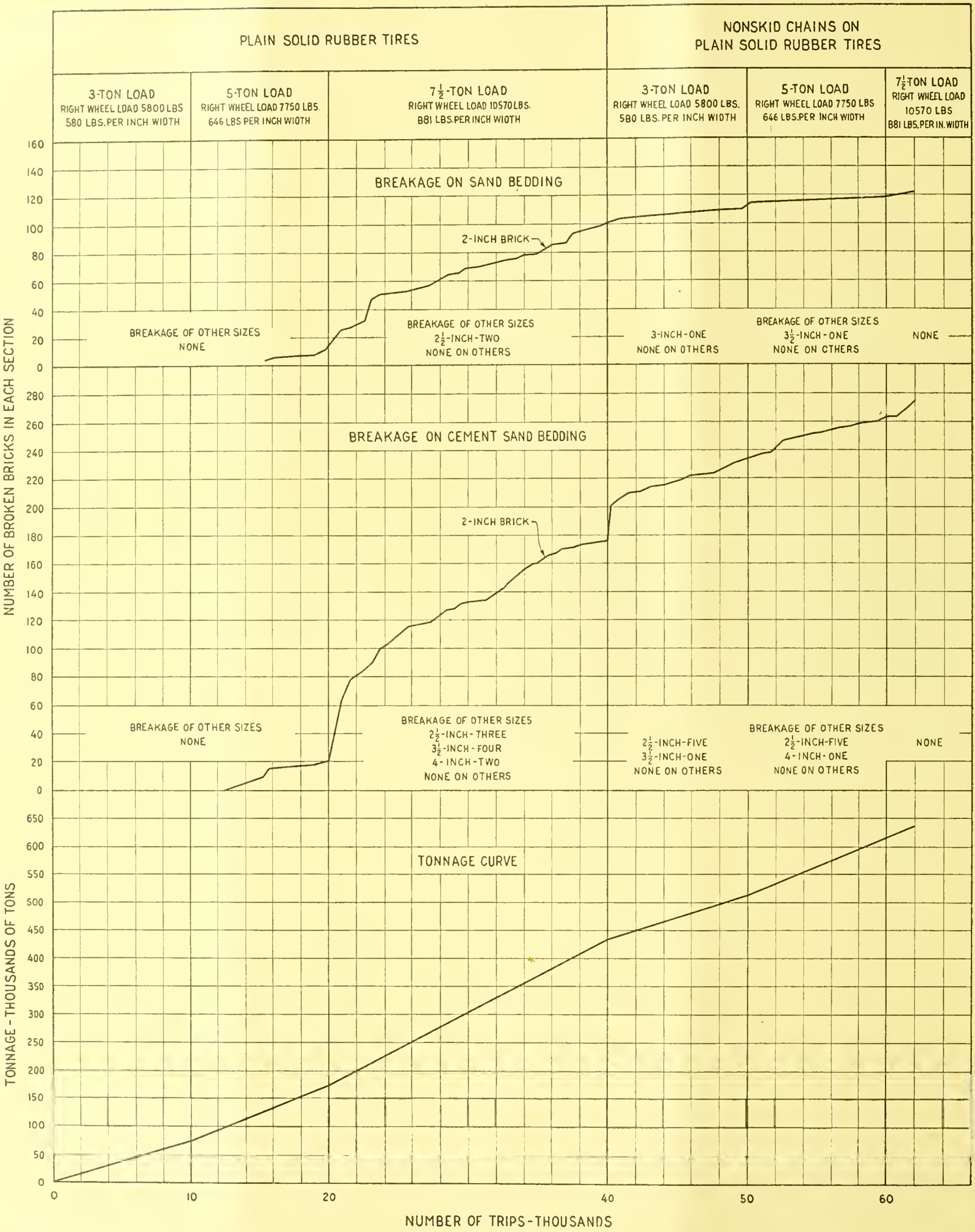


Fig. 5.—Results of the Accelerated Traffic Test as Measured by the Number of Brick in the 30-Inch Wheel Strips of Each Section, Broken Transversely Under Each Load in the Two Phases of the Test; and Curve Showing the Tonnage to Which the Brick Were Subjected

the remarkable strength of this brick under severe traffic conditions.

The marked contrast in breakage occurring on the two types of bedding course strikingly demonstrates the superiority of the plain sand over the cement-sand bedding, at least for the heavy-traffic pavements.

The higher rate of breakage which occurred during the early traffic, as shown in Table 2 and Figure 5, is probably explained by variation in the quality of the brick. It is probable that there were in each section certain brick slightly warped or of poorer grade, and these, breaking under the lighter loads, would tend to increase the early rate of breakage.

The greater breakage occurring under the outer wheels in the tests made with plain solid tires is shown by Table 5. This is the condition previously referred to and attributed to the fact that the trucks when equipped with plain solid tires were operated at a speed of 12 miles per hour whereas the pavement was superelevated for a speed of 9 miles per hour. The greater load which, under this condition, would be thrown upon the outer wheel, is doubtless the cause of the greater breakage which is shown by Table 5 to have occurred in the outer wheel strip under the plain solid-tired traffic. In the tests made with nonskid chains on the rear wheels the speed of the trucks was reduced to 9 miles per hour and it will be seen from the table that the breakage in the two wheel strips in this phase of the test was more nearly equal.

Table 5.—*Transverse breakage occurring in the 2-inch brick sections under the outer and inner wheels*

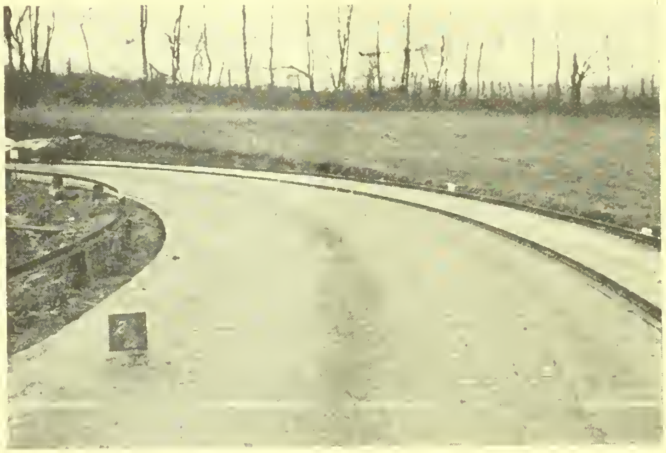
Tire condition	Speed	Rated load	Number of trips	Number of brick broken transversely	
				Inner wheel	Outer wheel
		Tons			
Plain solid tires	12 miles per hour	3	10,000	0	0
		5	10,000	2	34
		7½	10,000	19	149
		7½	10,000	10	63
Nonskid chains on plain solid tires.	9 miles per hour	3	10,000	45	18
		5	10,000	20	22
		7½	2,200	12	4

CONDITION OF THE TEST PAVEMENT AFTER COMPLETION OF ACCELERATED TESTS

Although the amount of transverse breakage has been taken as the criterion of the relative service of the several thicknesses of brick it must be understood that the transverse breakage alone did not materially affect the condition of the pavements. The broken portions remained in position, and under the plain-solid-tired traffic did not ravel or scale at the cracks except in the 2-inch sections. The number of brick broken during the entire test was less than the number that would ordinarily be broken during the rolling of a brick pavement.

Figures 6 to 10, inclusive, show the condition of the several brick sections at the completion of the plain-solid-tired traffic; and it will be observed that the damage in all sections was practically limited to transverse breakage. Practically all the cobbling resulted from the operation of the traffic equipped with nonskid chains, and the greater part occurred during the early stages of this traffic. It seemed that after the corners of the brick had become slightly rounded, further rounding took place very slowly. The spacing between the brick appeared to be a controlling factor in the amount of cobbling which took place; and brick laid

2-Inch Brick on Sand Bedding


2-Inch Brick on Sand-Cement Bedding

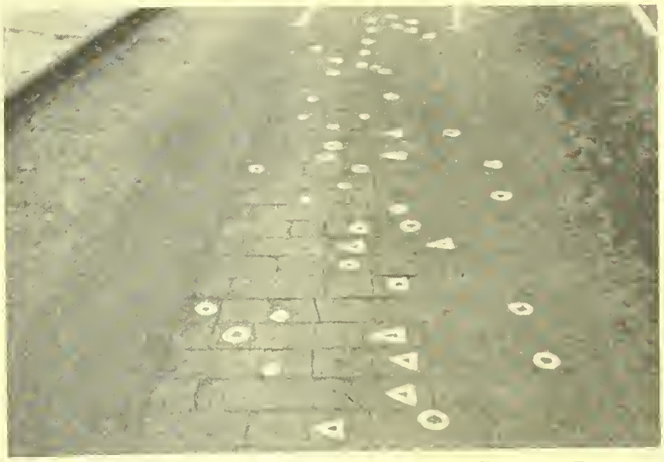

Close-up View of 2-inch Brick on Sand-Cement Bedding

Fig. 6.—Condition of the 2-Inch Brick Surface After the Completion of the Plain-Solid-Tired Traffic. Broken Brick are Marked with White Painted Symbols the Shape of Which Indicates the Amount of Traffic Which has Passed Over the Section Up to the Time the Break Occurred. For Explanation of Symbols see Table 4

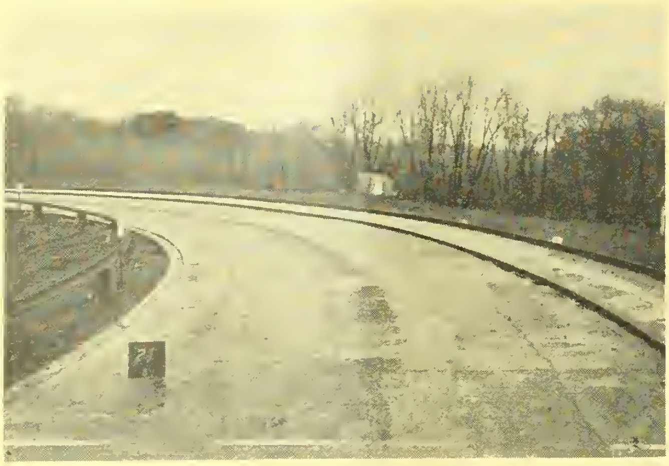


FIG. 7.—CONDITION OF THE 2½-INCH BRICK SECTION LAID ON SAND BEDDING AFTER COMPLETION OF THE PLAIN-SOLID-TIRED TRAFFIC

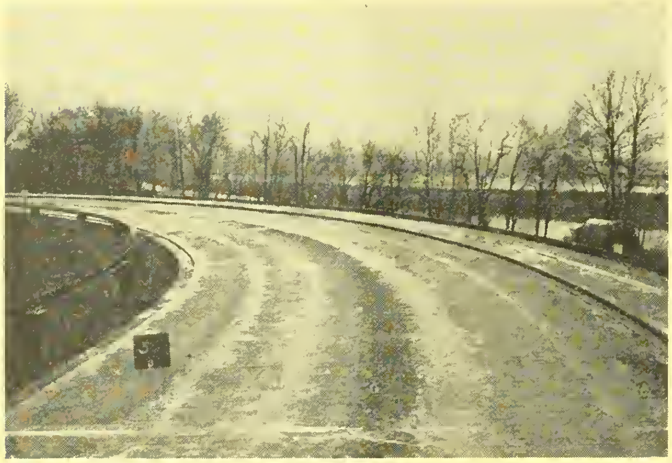


FIG. 8.—CONDITION OF THE 3-INCH BRICK SECTION AFTER COMPLETION OF THE PLAIN-SOLID-TIRED TRAFFIC. THERE WAS NO APPARENT DIFFERENCE BETWEEN THE SECTIONS ON SAND AND CEMENT-SAND BEDDING COURSES

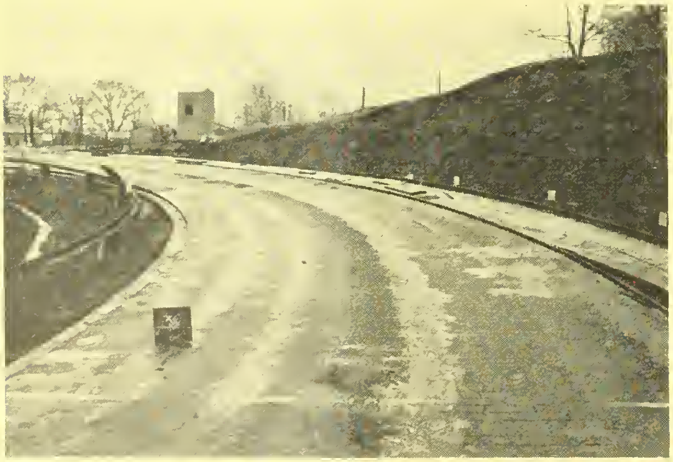


FIG. 9.—CONDITION OF THE 3½-INCH BRICK SECTION AFTER COMPLETION OF THE PLAIN-SOLID-TIRED TRAFFIC. THERE WAS NO APPARENT DIFFERENCE IN THE SECTIONS LAID ON THE TWO BEDDING COURSES

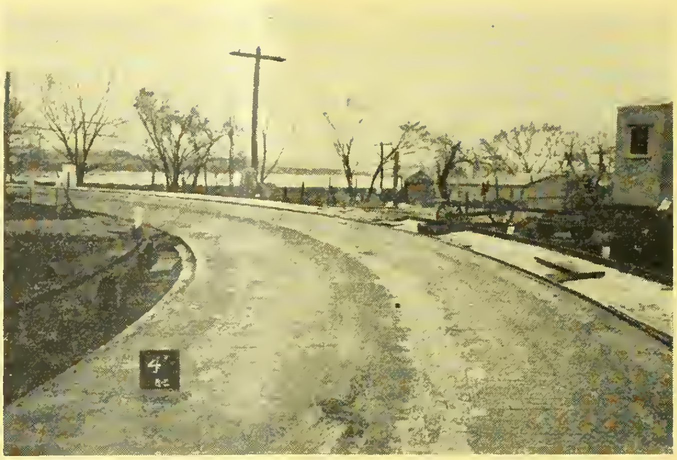


FIG. 10.—CONDITION OF THE 4-INCH BRICK SECTION. IN THIS CASE ALSO THERE WAS NO APPARENT DIFFERENCE BETWEEN THE SECTIONS ON THE TWO BEDDING COURSES

with a wide spacing consistently showed greater cobbling. Edges of the cracks formed in broken bricks were rounded about the same amount as the original edges of the brick.

The condition of the brick in the several sections after the completion of the chain-equipped traffic is shown in Figures 11, 12, and 13. It will be noted that very little difference is apparent between the condition of the 2½, 3, 3½, and 4 inch brick sections, all being in almost perfect condition at the completion of the traffic test. The sections of 2-inch brick, on the other hand, show marked effects of the heavy traffic.

TEST TRAFFIC COMPARED WITH TRAFFIC ON ACTUAL HIGHWAYS

It will be seen from the tonnage curve in Figure 5 that a total of 62,200 trucks passed over the 30-inch strips of the test pavements, that about one-third of these were equipped with the heavy nonskid chains, and that the total tonnage moved during the period of the test amounted to approximately 630,000 tons.

A quantitative comparison of the traffic applied to the test sections with the actual traffic using a few heavy-traffic highways is made possible by the studies previously made by the Bureau of Public Roads of the transverse distribution of truck traffic on paved highways of various widths.[1] Using the data from these studies it is possible to estimate the maximum concentration of actual traffic on a strip of any given width in terms of a percentage of the total traffic. Applying these percentages to the known traffic on certain heavily traveled highways it is possible to approximate the maximum amount of traffic passing over a strip of these highways 2½ feet wide corresponding to the width of the traffic lanes on the test track.

For purposes of comparison in this manner certain highways in Cook County, Ill., have been selected. According to the survey of traffic on the highways of the county made by the Bureau of Public Roads and local authorities,[2] the truck traffic on these highways, shown in Table 6, is the heaviest to be found on any highways in the county. It is interesting to note, therefore, that the traffic of trucks of the several capacities applied to the test track is equivalent to the corresponding traffic over these highways in periods of from 2 to 146 years, as shown by Table 7. In terms of total tonnage the test traffic is equivalent to five years of traffic on the most heavily traveled of these important motor truck arteries.

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[1] Transverse Distribution of Motor Vehicle Traffic on Paved Highways, by J. T. Pauls, PUBLIC ROADS, vol. 6, No. 1, March, 1925.

[2] Highway Traffic and the Highway System of Cook County, Ill., by the Bureau of Public Roads and the Cook County Highway Department.

Table 6.—*Daily loaded truck traffic on several highways in Cook County, Ill., 1925*

Name of highway	Width of pavement	Capacity of trucks						Total tonnage
		3 to 4 tons		5 to 5½ tons		6 to 7½ tons		
		No.	Tonnage	No.	Tonnage	No.	Tonnage	
	Feet							
Waukegan Road	18	19	143	18	190	8	120	450
Do	18	20	150	57	599	3	45	794
Lincoln Avenue	18	14	105	23	242	9	135	482
Dempster Street	18	3	23	6	63	1	15	101
Halsted Street	18	32	240	35	368	1	15	623
Ogden Avenue	24	76	570	29	305	3	45	920

Table 7.—*Length of time required for selected pavements in Cook County, Ill., to carry on two 30-inch strips, traffic in number of vehicles and tonnage equivalent to that applied to the test pavement*

Name of highway	Capacity of trucks						By total tonage
	3 to 4 tons		5 to 5½ tons		6 to 7½ tons		
	By trips	By tonnage	By trips	By tonnage	By trips	By tonnage	
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
Waukegan Road	7	7	7	7	18	18	9
Waukegan Road	7	7	2	2	48	48	5
Lincoln Avenue	9	9	6	6	16	16	9
Dempster Street	44	44	22	22	146	146	45
Halsted Street	4	4	4	4	146	146	7
Ogden Avenue	2	2	5	5	57	57	5

In the above evaluation of the traffic applied to the test sections only a few of the most important truck-carrying highways were used for comparison. A more equitable comparison would probably be obtained were we to use the average of the truck traffic on the Cook County highway system. Compared in this way the traffic on the test sections is shown by Table 8 to be equivalent to the average traffic using all highways in the county in 18 years, and the test traffic of 6 to 7½ ton trucks to be equivalent to the corresponding actual in 70 years.

Table 8.—*Average trips and tonnage of the several sizes of trucks passing over the highways of Cook County, Ill., and the time value of the test traffic, based on an 18-foot two-way pavement*

Capacity of trucks	Average truck traffic on Cook County highways		Duration of actual traffic equivalent to that on test sections
	Trucks per day	Approximate tonnage	
			Years
3 to 4 tons	16	121	8
5 to 5½ tons	9	95	14
6 to 7½ tons	2	30	70
All capacities	27	246	18

Both the plain-tired and the nonskid-chain-equipped traffic applied to the test pavement was used in making the comparison shown in Tables 7 and 8. To obtain a comparison of that portion of the test traffic equipped with chains with similar traffic on actual highways, it is necessary to make an estimate of the portion of the year during which such traffic passes over the highways. It is believed that by estimating the yearly duration of this type of traffic at two months, ample time is allowed to cover the most severe conditions. On this or any other reasonable basis of comparison it will readily be seen that the test traffic with chains was far greater than any that could possibly come on any actual highway during the life of the pavement.

THE RESULTS OF THE PHYSICAL TESTS OF THE BRICK

The quality of the brick used in the several test sections is shown by the results of the physical tests recorded in Table 3. These tests indicate that the brick used in the test pavement were of about average quality, though some difference appears to exist in the quality of the different sizes.

Rated according to their moduli of rupture and crushing strength the test results indicate the order in quality of the different sizes to be as follows: 2½-inch (best); 3½-inch; 4-inch; 3-inch; 2-inch (poorest).

The rattler losses, on their face, indicate a different order; but it has long been recognized that for brick of equal quality but different in size, the comparative rattler losses are not directly proportional to the differences in weight. Engineers have recognized the injustice of specifying the same percentage of wear for different thicknesses, and have adopted, very generally, the practice of allowing some tolerance for the thinner brick in comparing their rattler loss with that specified for the thicker brick.

It is apparent that the loss in the brick undergoing the rattler test is more nearly a function of the total length of edge than of the weight of the brick. In view of the increasing use of the thinner brick, it would seem, therefore, that some modification in the standard rattler test should be made so as to make it applicable to sizes of brick differing from that of the original standard block. Until such a modification is adopted, however, it will be necessary to correct the observed losses for the various sizes of brick so as to convert them to a comparable basis.

Applying the corrections recently suggested by the Bureau of Public Roads,[3] to the rattler losses of the various sizes as shown in Table 3, the order of quality of the different sizes of brick used in these tests is brought into conformity with that indicated by the modulus of rupture and crushing strength tests.

The results of the compression test made on the brick taken from the test pavement after the completion of all traffic are given in Table 4. In making this test it was hoped to correlate the result of this test with service behavior. Comparing the crushing strength of the different brick with their condition after the test, it will be noted generally that good conditions both as to rounding and breakage were characteristic of the bricks giving the higher crushing results, while broken or badly rounded brick gave consistently lower values.

A striking point noticed in the data from the compression tests on the brick which had carried traffic was the consistently lower value obtained from this test as compared with the result of the same test on brick that had not been subjected to the heavy test traffic. It was first thought that this might be an indication of fatigue in those brick which had carried the heavy continuous traffic of the test. In order to obtain more data on this point additional tests were made. At first, a series of check compression tests was run. These test data, although consistent in themselves, did not agree with those obtained in the former series of tests. It appeared that the difficulty might be, and probably was, due to the failure of the capping on the worn brick. Accordingly transverse bending tests were run on brick which had carried traffic and on those which had not and these results did not show the reduction in strength indicated by the compression tests.

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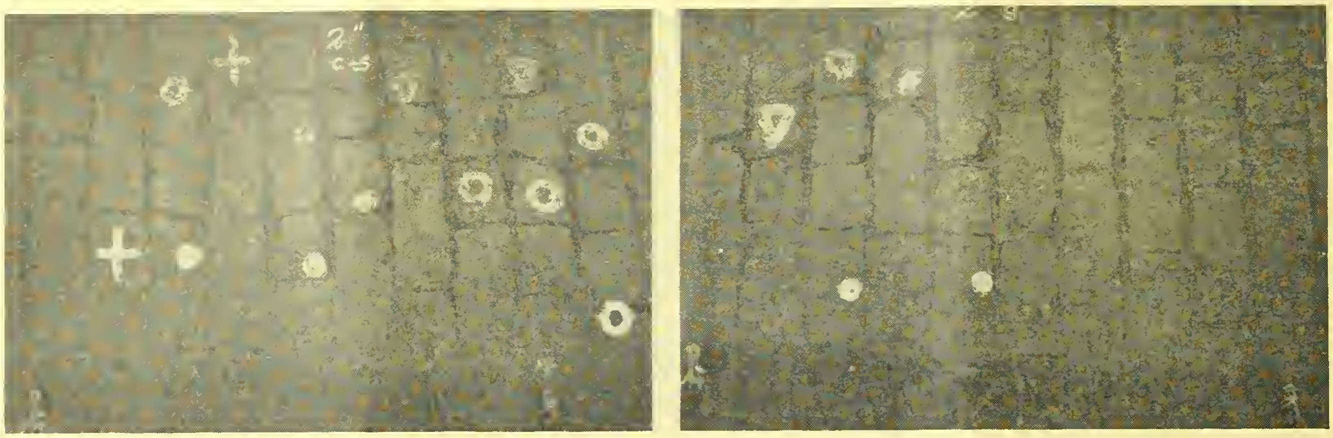
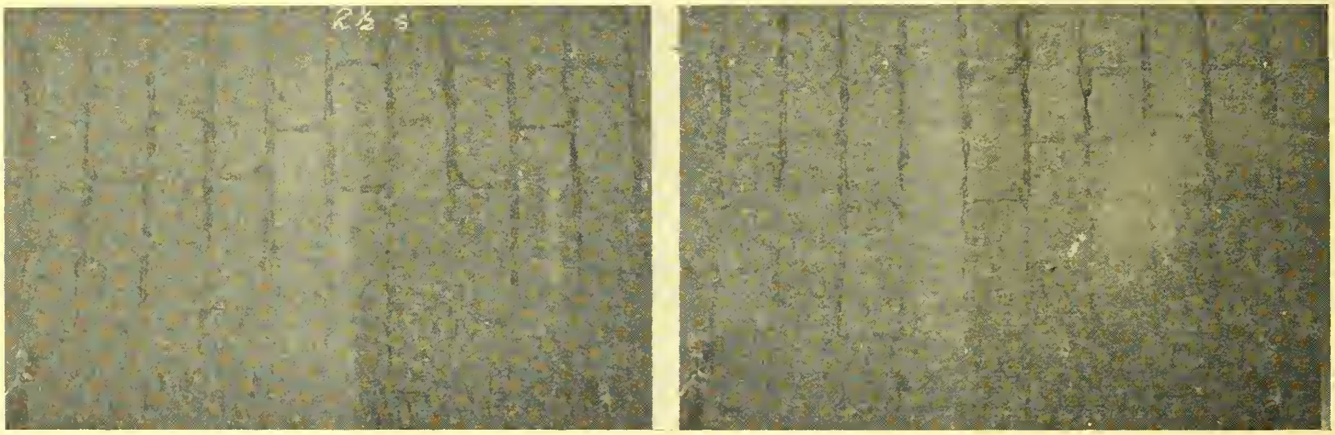
[3] Effect of Size of Brick on Rattler Loss, by F. H. Jackson, Public Roads, vol. 7, No. 5, July, 1926.



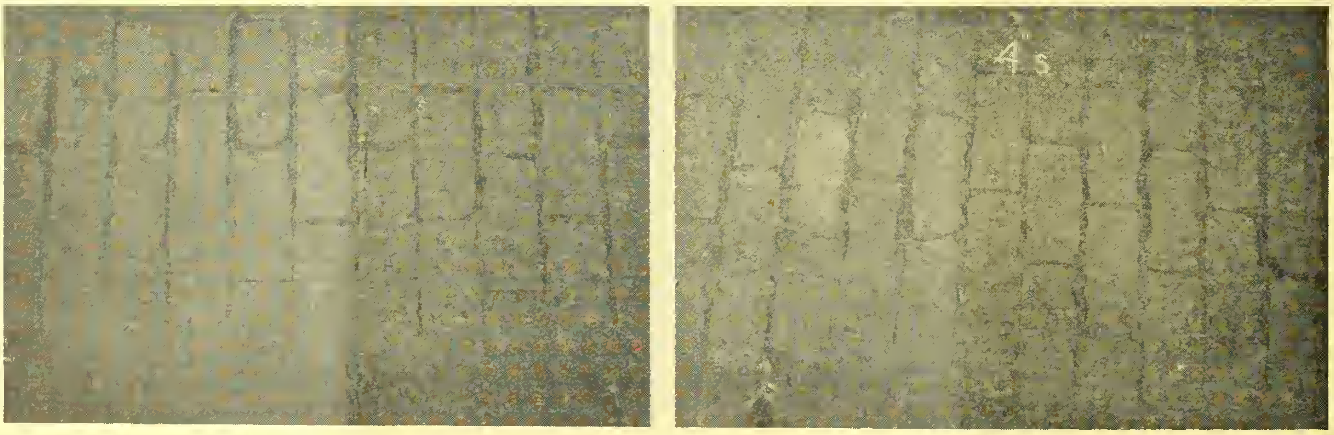
2-Inch Brick or Sand-Cement Bedding

2-Inch Brick or Sand Bedding



2½-Inch Brick

3-Inch Brick



3½-Inch Brick

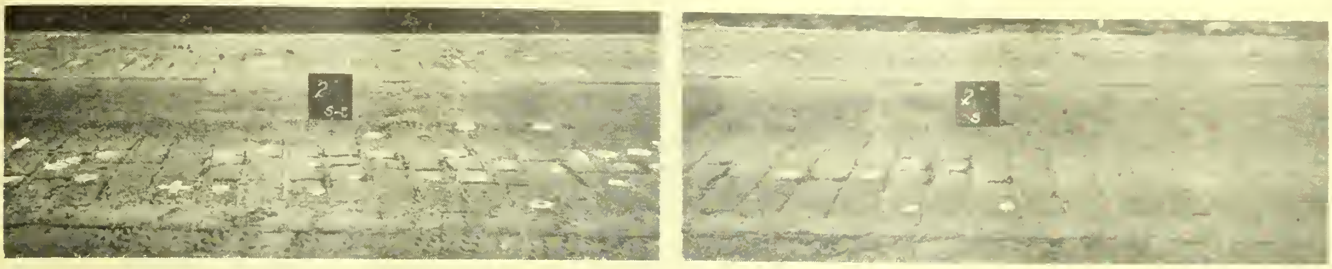
4-Inch Brick

Fig. 11.—Looking Down on the Several Sections After Completion of the Chain-Equipped Traffic. None but the 2-Inch Brick Showed Sufficient Breakage to Justify a Conclusion as to the Effect of the Type of Bedding. For Explanation of Symbols see Table 4

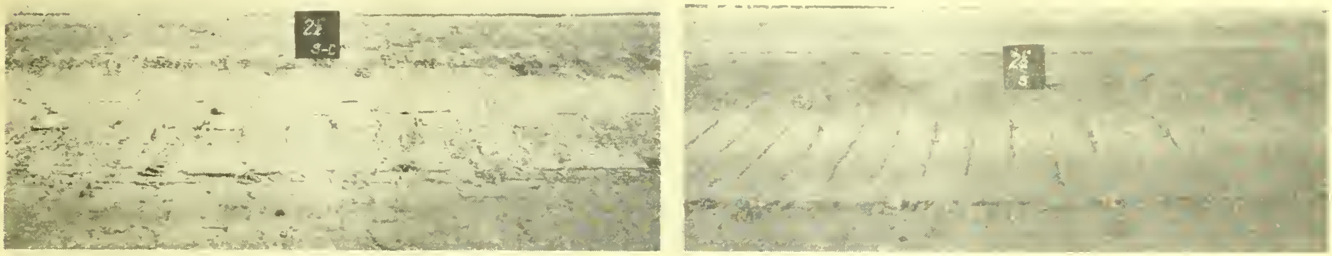
THE FIELD SURVEY

By making a condition survey of thin-brick pavements in actual service, and by obtaining information from local engineers and highway officials regarding such factors as construction, age, climatic conditions, traffic, and maintenance, it has been possible to arrive at some conclusion as to the merits of the thinner paving brick in actual service.

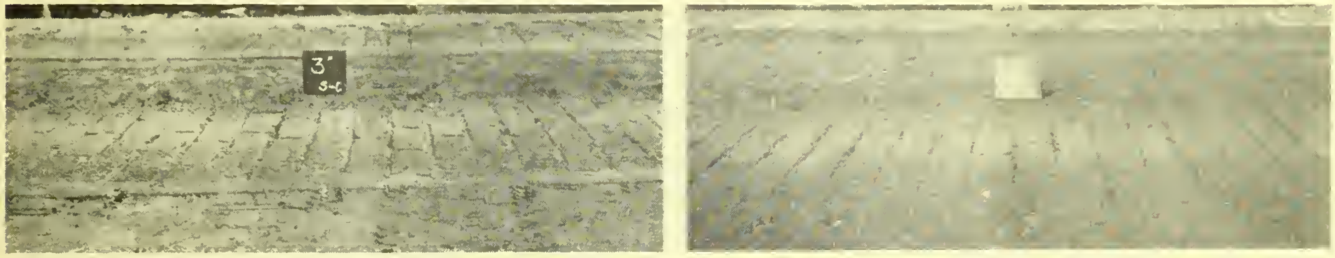
Pavements in which brick less than 3 inches in depth have been in service for a considerable length of time are practically limited to portions of Texas, Louisiana, Oklahoma, and Nebraska. The field survey was thus, of necessity, confined to this section. Every effort was made to obtain, through consultation with local engineers and highway officials, accurate information regarding all conditions which might influence the


2-Inch Brick on Sand-Cement Bedding

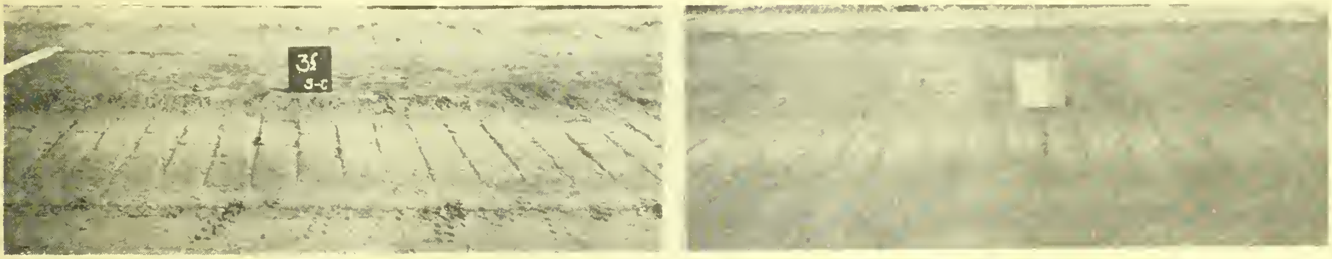
2-Inch Brick on Sand Bedding


2½-Inch Brick on Sand-Cement Bedding

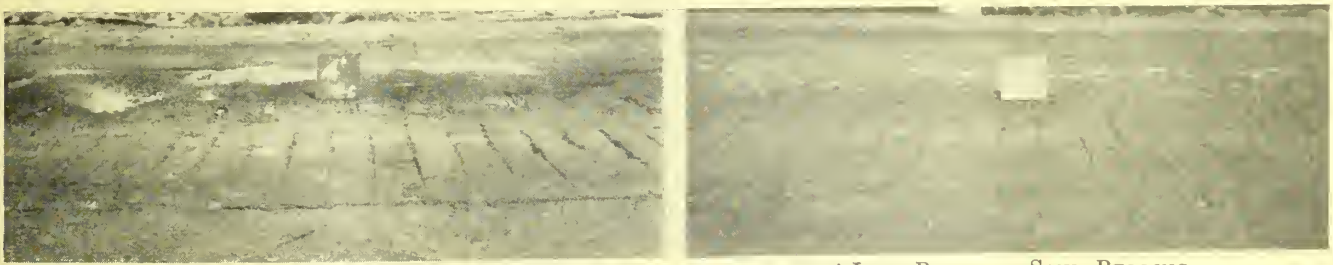
2½-Inch Brick on Sand Bedding


3-Inch Brick on Sand-Cement Bedding

3-Inch Brick on Sand Bedding


3½-Inch Brick on Sand-Cement Bedding

3½-Inch Brick on Sand Bedding


4-Inch Brick on Sand-Cement Bedding

4-Inch Brick on Sand Bedding

Fig. 12.—Looking Across the Several Sections After Completion of the Chain-Tired Traffic, Showing the Breakage and Degree of Cobbling

behavior of each pavement, and also their views as to the adequacy of the particular pavement to meet the traffic requirements.

Special attention was paid to the effect of the first attempt at using thinner brick on the subsequent policy with regard to brick thickness, using this criterion as a measure of the sufficiency of the design. The first thin brick pavement laid in a community in many cases can be classed as an experiment but similar construction later may be taken as an expression of the satisfaction of the community with the type of construction.

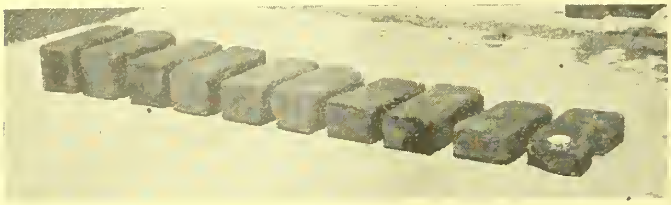


Fig. 13.—Brick of the Different Sizes Taken from the Test Pavement After Traffic. The Group Shows the Extreme Variations in the Condition of the Brick After the Service Tests

This survey involved a detailed inspection of the condition of several million square yards of pavements in which brick of 2½ and 2¼ inch thickness were used. Data on age, type of construction, type of traffic, maintenance, and other influencing factors were obtained for each pavement inspected.

OBSERVATIONS MADE DURING THE FIELD STUDY

The following paragraphs give condensed information with regard to the condition of the brick pavements inspected which were built with brick under 3 inches in thickness. The condition reported is that which existed in February, 1926.

Greenville, Tex.—Located in the black waxy soil area, an unusually bad subgrade condition. This was the first city to lay brick flat, and to use bricks of 2¼-inch thickness.

Approximately 12,000 square yards of 2¼-inch repressed brick was laid on Main Street during 1905–06. The brick were laid on a 5-inch pit-run-gravel concrete base, and 1½-inch sand cushion with a filler of 1:2 grout.

This pavement has had heavy traffic, a large amount of which has been in steel-tired vehicles. Very little breaking, rounding, or cobbling has taken place, the pavement being in excellent condition except at a few places where breaks in the base have occurred (fig. 14).

Approximately 90,000 square yards of brick pavement was laid during 1914, using 2½-inch wire-cut, vertical-fiber brick. The construction consisted of a 4-inch concrete (1:7 pit-run gravel) base, 1½-inch sand bedding, and asphalt filler (penetration 57).

These pavements were in nearly perfect condition although they had received no maintenance except at a few locations where failures in the base, brought on by the unusual, unstable subgrade, had occurred.

Sulphur Springs, Tex.—Better subgrade conditions than at Greenville. Paved streets carry traffic occasioned by surrounding rich agricultural area (fig. 15).

Approximately 60,000 square yards of 2½-inch brick were laid on Main Street and Square during 1915. The construction was 4-inch concrete (1:7 pit-run gravel) base, 1-inch sand bedding, and asphalt filler (penetration 57).

The condition of these pavements was nearly perfect; very few brick had broken and very little cobbling had taken place. They had received no maintenance except on a short portion of Main Street where additional asphalt filler had been recently applied.

Connolly Street paved with 3-inch brick during 1915 appeared to be in no better condition than the 2½-inch brick laid during the same year.

Tyler, Tex.—This city is located in the iron-ore belt. The soil should make a very stable subgrade for any type of pavement. The traffic on some of the strreets is fairly heavy.

A considerable number of 3-inch brick were laid during 1920 on a natural soil base and some on old water-bound macadam surface. Both types of pavement were in nearly perfect condition, although they had had no maintenance prior to the date of inspection.

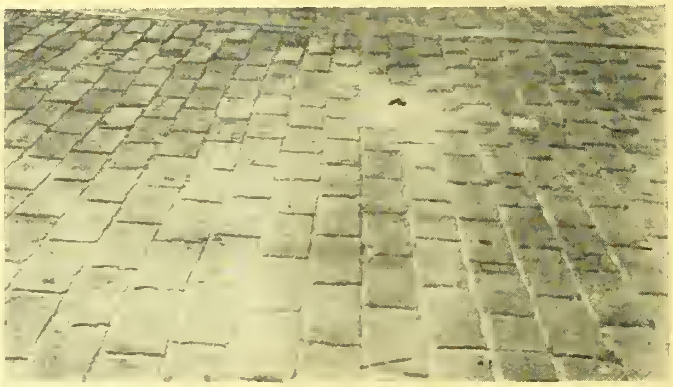


Fig. 14.—Main Street, Greenville, Tex. Detail View of the 2¼-Inch Repressed Brick Surface Laid 1905 06

When the inspections were being made 140,000 square yards of 2½-inch brick pavement was being laid on several streets of the city, the new pavements being laid on 4-inch concrete (1:6 pit-run gravel) base, with a 1-inch sand bedding and asphalt filler.

The Cotton Belt Railroad has built 2½-inch brick pavements on 6-inch concrete base along the unloading tracks in its freight yards.

Temple, Tex.—This city is in the black waxy soil area. The surrounding country is a rich, agricultural district and traffic on many of the city streets is heavy.

Approximately 40,000 square yards of 2½-inch brick were laid on Main and other streets during 1915. The construction was as follows: 4 to 5 inches of concrete (1:8 pit-run gravel) base, 1-inch sand bedding and asphalt filler. The condition of these pavements at the time of the inspection was good, although some failures had occurred in the bases apparently from bad subgrade conditions (fig. 16).

From 1923 to 1925, 60,000 square yards of 2½-inch brick were laid, the construction being the same as in 1915, using the 4-inch concrete base on residential streets and 5 inches on business streets.

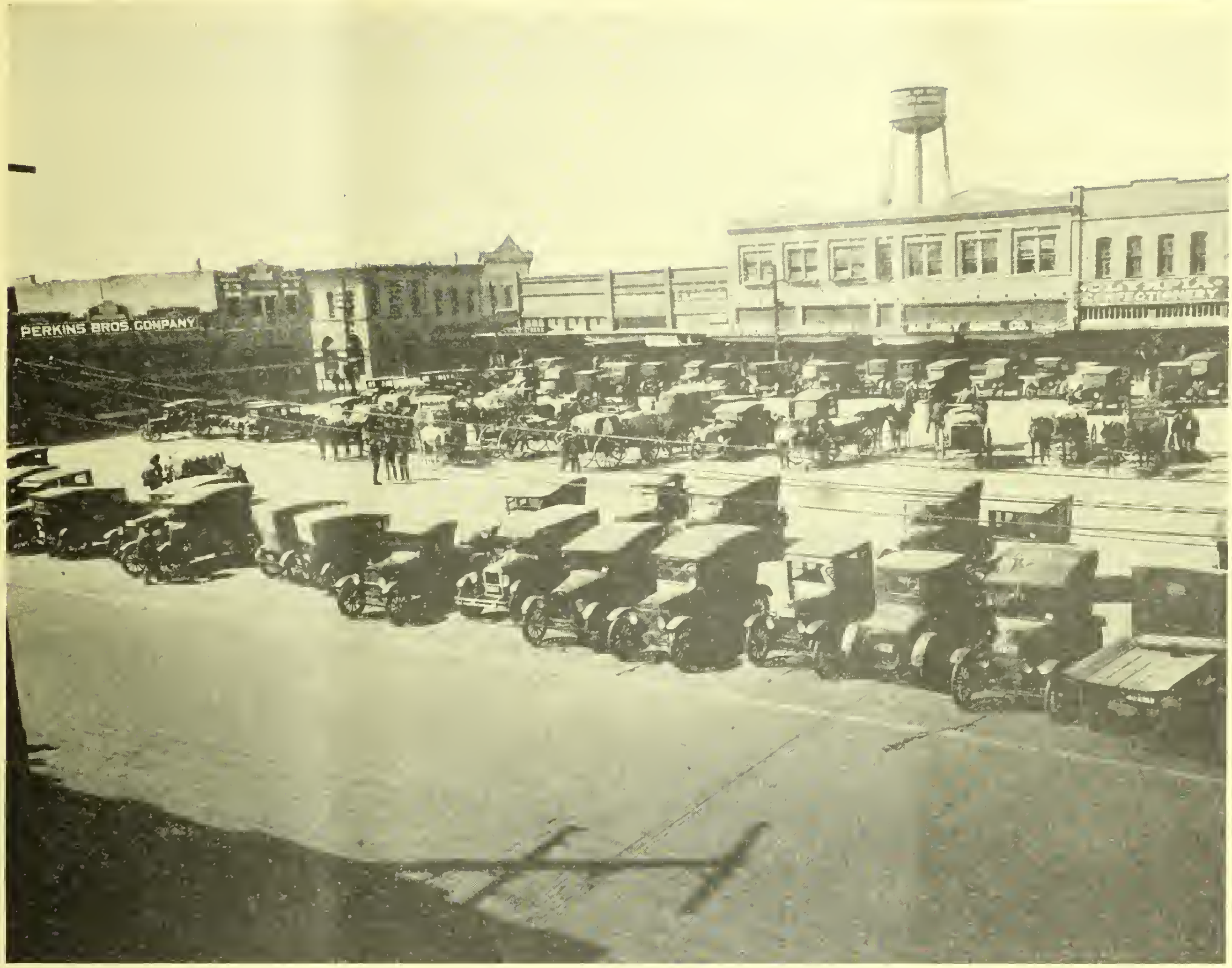


FIG. 15.—PUBLIC SQUARE, SULPHUR SPRINGS, TEX. TWO AND ONE-HALF-INCH BRICK LAID IN 1915 ON A 4-INCH CONCRETE BASE WITH 1-INCH SAND BEDDING AND ASPHALT FILLER

Prior to 1923 some of the pavements constructed were built of 3-inch brick; since then only the 2½-inch size has been used. No difference in the condition of streets paved with the 3 and 2½-inch brick was apparent at the time of the inspection.

Fort Worth, Tex.—This city is using 2½-inch brick on residential and outer business streets. The construction is 5-inch (1:3:5) concrete base, 1-inch sand bedding and asphalt filler.

Victory Boulevard was paved during 1925 with 2½-inch brick, laid on a 5-inch (1:3:5) concrete base with 1-inch (1:4 cement-sand) bedding and asphalt, filler (penetration 57). This street has a very heavy truck traffic to and from the wholesale district of the city. The condition of the pavement was nearly perfect when inspected and no ill effect of the heavy truck traffic was visible.

Okmulgee, Okla.—A great deal of oil field traffic is carried over the city streets. The old brick pavements, in particular, have been subjected to a large amount of steel-tired traffic.

From 1916 to 1923, 220,000 square yards of 2½-inch brick was laid on 4½ to 5 inch (1:3:5) concrete base, with 1½-inch sand bedding and asphalt filler. Most of these pavements were in good condition when inspected and had required no surface maintenance. The portion built during 1916 was of a local and inferior brick and it had cracked and cobbled badly.

About 30,000 square yards of 3-inch brick was laid during 1921. None of this size has been laid since.

Henryetta, Okla.—The streets in this city carry heavy traffic to and from the surrounding oil and coal fields.

During 1918, 1920, 1922, and 1923, 200,000 square yards of 2½-inch brick were laid on 5-inch (1:3:5) concrete base, with 1-inch bedding and asphalt filler. These pavements were in perfect condition and had received no surface maintenance.

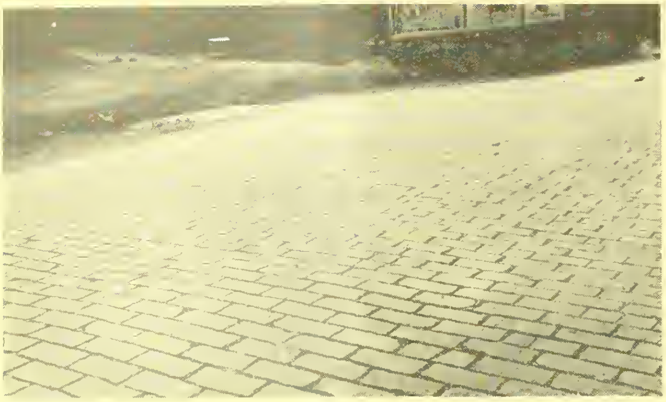
About 25,000 square yards of 3-inch brick were laid during 1917. This portion had been refilled with asphalt; and no brick of this thickness had since been laid.

Wetumka, Okla.—The streets carry heavy oil and cotton traffic.

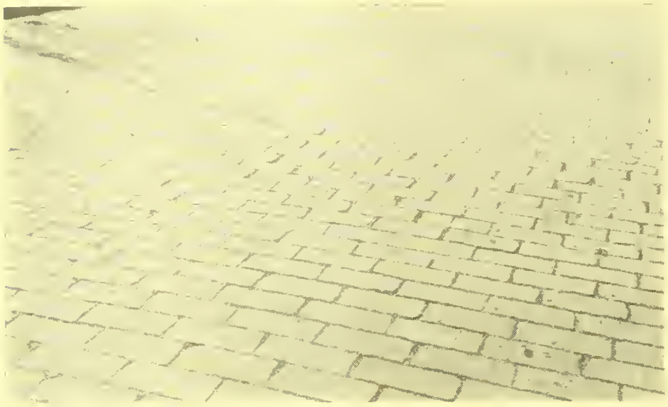
About 12,000 square yards of 3-inch brick and 50,000 square yards of 2½-inch were laid during 1924 and 1925. The construction was: 5-inch (1:3:5) concrete base, 1-inch sand bedding and asphalt filler (penetration 55). An excessive quantity of filler was used during construction and was still on the surface when inspected. Shoving and peeling of this mat will very likely cause a rough surface.

Okeemah, Okla.—The principal streets carry heavy traffic from the surrounding oil fields.

About 35,000 square yards of 2½-inch brick were laid during 1920. The construction was: 5-inch (1:3:5) concrete base, 1-inch sand bedding, grout



Main Street—2½-Inch Brick on a 5-Inch Concrete Base, 1-Inch Sand Bedding and Asphalt Filler



Residential Street—2½-Inch Brick on a 4-Inch Concrete Base, 1-Inch Sand Bedding and Asphalt Filler

Fig. 16.—Condition of Brick Laid at Temple, Tex., in 1915 on Main Street and a Residential Street

filler on a portion of Main Street and asphalt on the other pavements. These streets were in nearly perfect condition when inspected in February 1926 (fig. 17).

Oklahoma City, Okla.—About one-half mile of 18-foot pavement was laid with 2½-inch brick during 1918. This pavement is on the main entrance to the city and carries very heavy traffic. The construction was: 4-inch (1:3:6) concrete base with monolithic 4-inch concrete curbs, 1-inch sand bedding, and asphalt filler. It was in nearly perfect condition, there being very few broken bricks and very little cobbling. It had had no surface maintenance (fig. 18).

Oklahoma County, Okla.—The county is building 5 miles of 18-foot brick pavement using 2½-inch brick on the same approach to Oklahoma City. The construction is: 5-inch (1:3:5) concrete base with 6-inch monolithic curbs. The base is being built with a floated finish, and 1-inch asphalt-filled transverse expansion joints are being put in the base about every 40 feet. A three-fourth inch thickness of sand bedding is used.

Ponca City, Okla.—Streets in this city have unusually heavy traffic because of the large oil and refining industry which centers in and around the city. The main streets are said to have greater traffic than any other city streets in Oklahoma.

Only 2½-inch brick have been used in this city, and of these about 500,000 square yards have been laid since 1919, including 50,000 square yards on Main street during 1919, 38,000 square yards during 1921 on an old 7-inch macadam surface, and 4,000 square yards on a new 7-inch macadam during 1924. All these pavements were in almost perfect condition and had had no maintenance up to the date of the inspection.

Prior to 1921 the pavements were laid on a 1:5 cement-sand mortar base. For this, in the pavements built during 1921 and later, there has been substituted a 4-inch (1:3:5) concrete base. In all pavements the bedding course has been 1 inch of sand, and the filler has been 39-penetration asphalt. These have been used with the macadam bases also. In preparing the old macadam a certain amount of shaping was necessary.

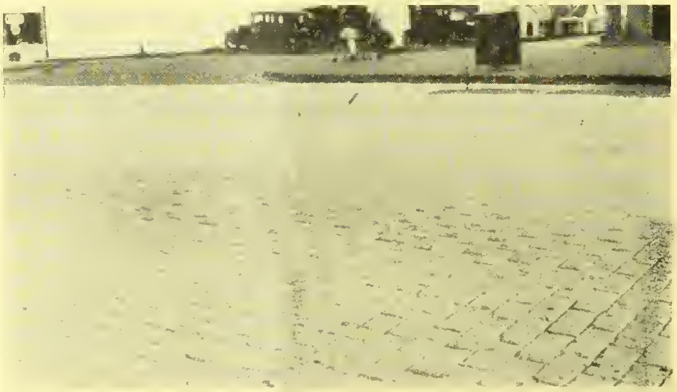


Fig. 17.—Detail View Showing Condition in February, 1926, of the 2½-Inch Brick Laid at Okeemah, Okla., in 1920 on a 5-Inch Concrete Base with Grout Filler

Fig. 18.—Detail View of the Condition in February, 1926, of the 2½-Inch Brick Laid on the Main Approach to Oklahoma City. The Brick were Laid in 1918 on a 4-Inch (1:3:6) Concrete Base and 1½-Inch Sand Bedding with Asphalt Filler

Tonkawa, Okla.—This city is in the oil-field area and the principal streets have very heavy traffic. About 35,000 square yards of 2½-inch brick were laid during 1919 on Main Street and about 75,000 square yards of the same size were laid during 1922 and 1924. The portion built during 1919 was laid on a 4-inch (1:4 cement-sand mortar) base, with 1½-inch sand bedding and 1:3 grout filler. Asphalt was used as a filler in the later work. Expansion joints were placed in the surface every 20 to 30 feet. No difference in the condition of the grout-filled and asphalt-filled surfaces was apparent (fig. 19).

Blackwell, Okla.—There is heavy traffic on all the principal streets.

About 1½ miles south of the city on the main highway north and south, 1,000 linear feet of 18-foot frozen concrete pavement was surfaced with 2½-inch brick during 1920. This surface is in perfect condition.

Two hundred thousand square yards of the same size brick were laid during 1914, the construction being 4-inch (1:5 cement-sand mortar) base, 1-inch sand bedding and cement-grout filler. Transverse expansion joints were put in the grout-filled surface every 40 feet.

During and since 1922, 125,000 square yards of 2½-inch brick have been laid, and in this construction asphalt (penetration 39) has been used in place of grout as a filler.

All these surfaces were in excellent condition when inspected, very few broken brick being noted.

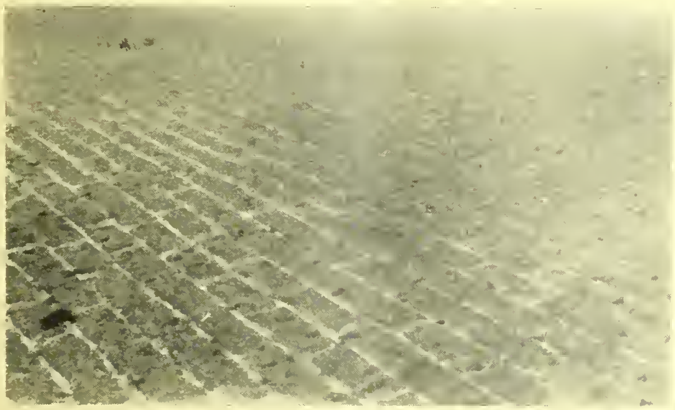


Fig. 19.—Detail View of 2½-Inch Brick Laid During 1919 on Main Street, Tonkawa, Okla. The Photograph Shows the Condition of this Pavement in February, 1926. It was Laid on a 4-Inch Mortar (1:4) Base and 1½-Inch Sand Bedding with Grout Filler

Texarkana, Tex.—Approximately 30,000 square yards of 2¼-inch brick were laid on the important streets during 1922. The construction was 6-inch concrete base, ¾-inch sand bedding and asphalt filler of penetration ranging from 54 to 45. An excessive quantity of filler was applied on practically all of this construction, the asphalt mat on many streets having a thickness of as much as one-half inch. Some peeling and shoving of this material has occurred in places, and further difficulties arising from this condition may be expected to occur for a long time.

A large number of 3-inch brick have been laid on 6 and 7 inch compacted gravel base. A thick mat of asphalt filler similar to that found on the thin brick pavement covered the surface at the time of inspection and similar peeling and shoving were observed.

All the brick pavements in this city have been laid during the last four years. At the time of this inspection there was nothing to indicate that the thinner brick was giving any less satisfactory service than the 3-inch type.

Omaha, Nebr.—This city laid 2½-inch brick on Parker Street during 1915. The construction was: 5-inch (1:3:6) concrete base, 1½-inch sand bedding and asphalt filler.

This street carries residential traffic. The surface was found to be in excellent condition, very few breaks and very little rounding having taken place. The only maintenance required prior to the inspection was occasioned by three blow-ups occurring in the base.

THE INDICATIONS OF THE FIELD STUDY SUMMARIZED

A steady growth in the use of brick of less than 3-inch depth is shown by the field survey. Numerous communities were found which have adopted the thinner brick for use on some or all of their streets. Although the earlier work may be classed as experimental, as indicated by the small quantities put down and by the type of street selected for paving, subsequent paving with the thin brick in larger quantities and on streets carrying heavier traffic may well be taken as an expression of the satisfaction of the community with pavements of this type.

Table 9 shows a summary of data from some of the cities covered by the field survey in which brick of less than 3-inch thickness are used almost exclusively, which indicate these tendencies.

The pavements built with 2½-inch brick, in most cases, were in good condition. In a few localities failures had occurred in the base causing displacement in the brick. It was particularly noted that in such cases displacement in the brick had taken place without breakage. This was found to be true also for the brick over transverse and longitudinal cracks. Extreme cases of base failures with effect on the brick are shown in Figures 20 and 21.

In general, the brick surfaces were found to have had very little maintenance, except at places where failure had occurred in the base. A portion of the brick pavement laid on Main Street, Henryetta, Okla., during 1917, was refilled with asphalt during 1925, as the original filler had been largely carried away by traffic. Oklahoma enforces the five-year maintenance clause on new paving work, yet the larger paving contractors declared, without exception, that no allowance is made or need be made in the estimated cost of a paving job to cover a difference between maintenance costs of 2½-inch and 3-inch brick.

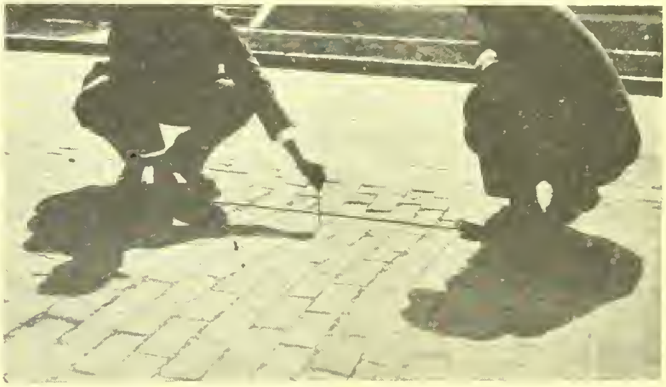


Fig. 20.—Condition of 2½-Inch Brick Surface Laid at Greenville, Tex., in 1914, as it Appeared in February, 1926. Displacement of the Brick has Occurred Over a Base Failure Without Breaking the Brick

It was found that excessive quantities of asphalt filler had been used in many sections of brick pavement. In one locality the excess covered the brick with a thickness of as much as one-half inch. Pavements where an excess of asphalt had been used were very rough, and will probably continue to be unsatisfactory for a long time. The importance of limiting the quantity of asphalt filler to that required to fill the space between the brick was strikingly illustrated by the unsatisfactory condition of such pavements, a typical example of which is shown in Figure 22.

TABLE 9.—*Summary of data of the field survey*

Location	Thickness of brick	Type of brick	Area laid	Year laid	Character of streets paved	Remarks
	Inches		*Sq. yds.*			
Greenville, Tex.	2¼	Repressed.	12,000	1905–06	Main	First brick to be laid flat.
Do	2½	Wire-cut vertical fiber.	90,000	1914	Main and residential.	No 3-inch brick laid.
Sulphur Springs, Tex.	2½	do	60,000	1915	do	
Do	3	do	(1)	1915	do	No 3-inch brick laid since.
Tyler, Tex	3	do	40,000	1920	do	Do.
Do	2½	do	140,000	1925	do	
Temple, Tex.	3	do	(1)	Before 1923	do	Do.
Do	2½	do	100,000	1915–1925	do	
Okmulgee, Okla	2½	do	220,000	1916–1923	do	
Do	3	do	30,000	Before 1921	do	Do.
Henryetta, Okla	3	do	25,000	1917	Main	Do.
Do	2½	do	200,000	1917–1923	Main and residential.	
Wetumka, Okla.	3	do	12,000	1924–25	Main	
Do	2½	do	50,000	1924–25	Residential.	
Okeemah, Okla.	2½	do	35,000	1920	Main and residential.	No 3-inch brick laid.
Ponca, Okla	2½	do	500,000	1919–1925	do	Do.
Tonkawa, Okla	2½	do	110,000	1919–1923	do	Do.
Blackwell, Okla	2½	do	325,000	1913–14	do	Do.

[1] Not known.

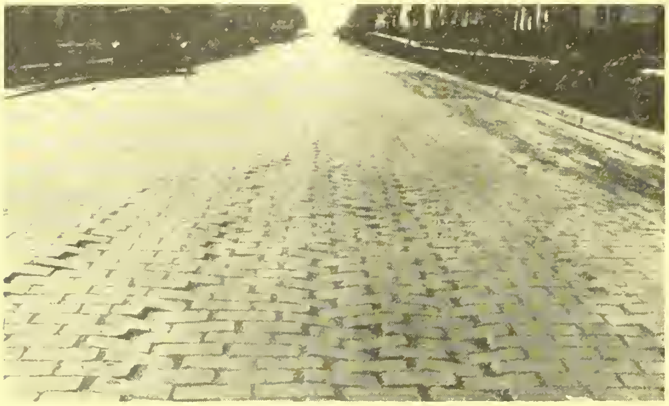


FIG. 21.—THE BASE FAILURE SHOWN IN THIS PHOTOGRAPH IS ON A RESIDENTIAL STREET OF TEMPLE, TEX. THE BRICK WERE LAID IN 1915 ON A 4-INCH CONCRETE BASE. THIS PAVEMENT IS IN THE BLACK WAXY SOIL AREA. NOTE THAT DISPLACEMENT OF THE BRICK HAS OCCURRED WITHOUT BREAKAGE

Some changes in the construction features of brick pavements have occurred since the earlier pavements were laid. The bedding course of the early surfaces was found to range in thickness from 1½ to 2 inches laid on a rough base. Later construction shows, generally, that a ¾-inch bedding has been used on a smoothly finished base. In a few localities very fine sand had been used for bedding, but, because this type of sand shows a tendency to work up between the brick and because of loss through cracks in the base, coarser sand has been substituted as being more satisfactory.

The field survey showed that, in general, brick laid on a thin bedding course with a smoothly finished base maintained a smoother surface than those laid with a greater depth of sand on a roughly finished base.

The research of the last few years has proved that the destructive effect of motor-truck impact is greatly reduced by the construction of smooth pavement surfaces. The continued perfect alignment of the finished brick surface is dependent to a large degree, particularly on heavy traffic thoroughfares, on the smoothness with which the base course is finished and the resulting uniformity of thickness and compaction of the bedding course. Any slight increase in the construction cost for the purpose of obtaining this condition will be more than justified by the potential increase in the life of the pavement.

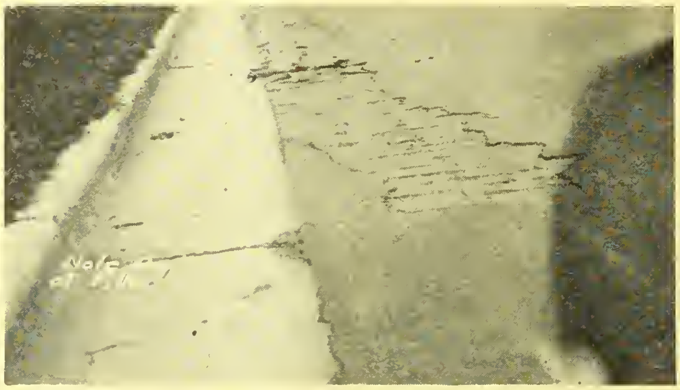


FIG. 22.—THE IMPORTANCE OF LIMITING THE QUANTITY OF ASPHALT FILLER TO THAT REQUIRED TO FILL THE SPACES BETWEEN THE BRICK IS ILLUSTRATED BY THE CONDITION OF THIS 2¼-INCH BRICK PAVEMENT IN TEXARKANA, TEX. THE PAVEMENT WAS LAID IN 1922

Cement-sand grout filler was found to have been used in the construction of some of the earlier brick pavements. Some of these were built with expansion joints in the brick surface, but in the majority of cases the joints have been omitted. The need for expansion joints with this type of filler was evidenced by the scaling in the surface of the brick on many of the pavements in which no expansion joints had been provided in the surface.

A wide range in the type and construction of base used was noted in the survey. Concrete ranging from 4 to 6 inches in thickness and largely of 1:3:5 proportion was found to have been used extensively in the area covered, while old and new macadam had been used satisfactorily in many instances. One city in the iron-ore belt was utilizing the iron-ore soil with entirely satisfactory results as base for brick pavements.

The successful use of these widely different types of base indicates that there is a wide range of possibilities in base construction. Any material which remains stable at all times would appear to make a satisfactory base for a brick pavement. It is evident that these requirements may be met in many cases by base construction of the less rigid type. As the function of the base is primarily to support the allowed wheel loads without appreciable vertical displacement, any type of construction that will meet this condition is satisfactory. An old macadam or other type of surface that has proved stable under traffic should prove entirely satisfactory as a base for brick.

In a majority of the locations covered in the field study no distinction was found to have been made in the base construction for brick of different thickness. One exception was noted where 3-inch brick had been laid on a compacted gravel base while concrete had been used for the 2¼-inch thickness. In this case and in others where difference in the base construction had been made with the use of brick of different thickness nothing was found in the condition of the pavements that would indicate the need for any variation in base construction for the different thicknesses of brick used.

Traffic.—A considerable number of the towns visited were located in the center of oil fields and as a result severe traffic conditions obtained on the more important streets. Formerly there was a considerable percentage of heavy steel-tired vehicles which have practically been superseded by heavy motor trucks. Although traffic records were not available for study it is believed that the traffic on the important streets of the cities visited was more severe than that on the streets of many of the larger cities.

A great deal of importance is attached, and properly so, to the views and experience of resident engineers and officials familiar with the use of brick under 3 inches in thickness. All of the officials interviewed from those sections of the country where brick of less than 3-inch thickness is being used expressed themselves as favorable to the use of the thinner brick, some with and others without limitations as to the type of street and traffic. Many maintained that 2½-inch brick would prove equally as satisfactory as the 3-inch thickness under all conditions, and others believed that the 2½-inch type should be limited to use on residential and outlying business streets.

The views of three prominent engineers who are intimately connected with the use of thin paving brick are expressed as follows:

> *Herman Beal, City Engineer, Omaha, Nebr.*—I have completed an investigation of a pavement in the city built during 1915 with a 2½-inch surface. The following information being available:
>
> Pavement on Parker Street between Twenty-ninth Street and Thirty-third Street, 30 feet wide, containing 5,360 square yards. Base 5 inches concrete, 1:3:6 mix. Bituminous filler.
>
> This pavement is in excellent condition, the brick showing practically no wear and the surface, with the exception of one or two expansion bulges, being exceptionally good. The traffic on this street is what might be classed as medium, although occasionally heavy truck loads are hauled thereon.
>
> Figure 23 is a detail view of the pavement described by Mr. Beal.
>
> *E. A. Kingsley, City Engineer, Dallas, Tex.*—We have used 3-inch vertical fiber brick for a number of years, and are using in our heavy traffic industrial district this character of brick in paving some of our new work this year. Our first 3-inch vertical fiber brick was laid on Market Street during 1919. This pavement has extremely heavy downtown industrial traffic of both the rubber and steel tired type. We have no trouble from breaking or cracking under traffic of the 3-inch brick. We have found that this brick carries traffic equally as well as did the old 4-inch block laid on edge. We can discover no reason in studying the wear and service given by the 3-inch brick which would suggest to us that a 3½-inch or 4-inch brick might be better.
>
> Dallas is perfectly satisfied with its heavy-traffic service on the 3-inch vertical fiber brick, and from experience that neighboring cities have had with the 2½-inch brick, we believe that this should be specified for the lighter traffic and residential districts.
>
> *D. Lewis, City Engineer, Fort Worth, Tex.*—This city has been laying 2½ and 3 inch vertical fiber brick with asphalt filler on a cement-sand cushion laid on a concrete base. We have found this type of paving to give excellent results.

MANUFACTURE OF THIN BRICK PRACTICABLE

The manufacture of brick as thin as 2½ inches is accomplished without particular difficulty. Some loss from warping was said to have occurred when brick of this thickness was first manufactured. Later changes in the manufacturing process, particularly in the burning, have made this loss negligible at the present time. The brick manufacturers in the sections of the country where the thinner brick are being used, were found to be favorable to the manufacture of the 2½-inch type. The manufacture of brick with 2-inch thickness was believed by some to be impractical because of the loss occurring from warping during burning.

Reduction in the cost of a brick pavement built of thin brick is the result of decreased cost of such items as manufacture, transportation, handling on the job, and saving in the filler material.

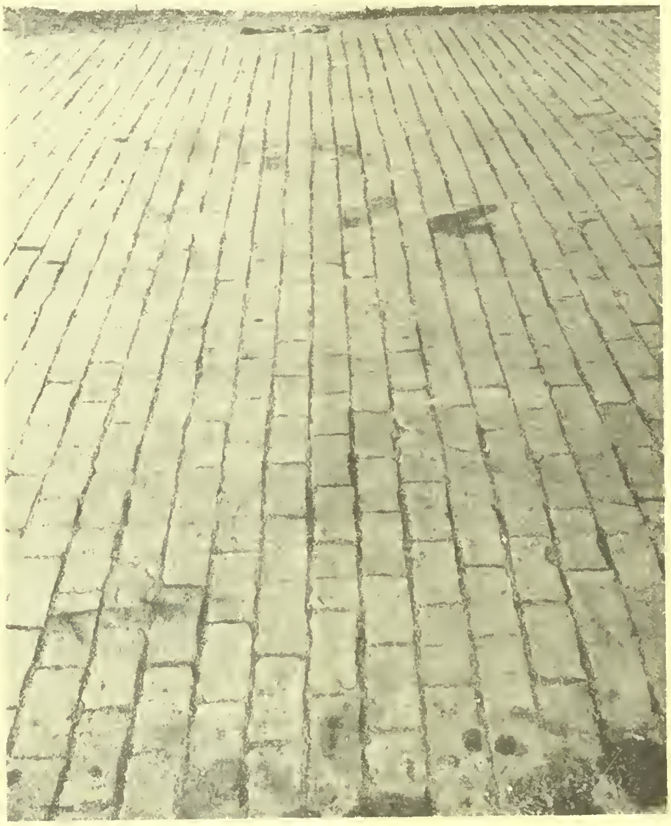


Fig. 23.—Detail View of 2½-Inch Brick Laid on Parker Street, Omaha, in 1915, Described by Herman Beal. The Brick were Laid on a 5-Inch Concrete Base (1:3:6) with Bituminous Filler. The Photograph Shows the Condition of These Brick in 1926 After 11 Years Under Medium Traffic

Based on the cost of a 3-inch pavement the saving in materials and costs for each ½-inch reduction in thickness would be approximately as follows:

	Cost reduction	Material reduction
	Per cent	*Per cent*
Manufacture	10	
Freight	16	
Haulage	16	
Filler	16	10

AN INSTRUMENT FOR THE MEASUREMENT OF RELATIVE ROAD ROUGHNESS

DEVELOPED BY THE UNITED STATES BUREAU OF PUBLIC ROADS

NO ARGUMENT is needed to convince the user of our roads of the desirability of as smooth a surface as can be produced. A rough surface gives rise to effects unpleasant to the passenger and detrimental to the vehicle. Smooth surfaces mean greater mileage with less fatigue, with less damage to cargoes, and with lower operating costs for the vehicle. Thus the subject assumes a very considerable economic aspect.

The highway engineer is vitally interested in building and maintaining smooth roads because of the effects already mentioned and further on account of the direct effect of surface roughness on the life of the pavement. The research of the last few years has clearly pointed out that road roughness produces impact and impact contributes to the early deterioration of any type of road surface. So convincing has been the evidence of this fact that State highway engineers have expressed the belief that smoothness is the most important quality to be sought in road surfaces.

The general appreciation of the importance of this matter by highway engineers has created a demand for an instrument with which the roughness of a road surface may be measured. With such an instrument the engineer would find it possible to specify the quality demanded in new work and to obtain accurate data on the deterioration of the surface in time to apply proper corrective measures. To be most useful such a device should not only be accurate but should supply the information rapidly and in such form that it can be used immediately. Finally, it should be mechanically simple and reliable and so designed that it can conveniently be used on an automobile.

Several devices have been offered from various sources to meet the demand. The Bureau of Public Roads experimented with a number of designs over a period of several years and finally, in May, 1925, developed one which when then used in the vicinity of Washington, D. C., appeared to be better in several respects than other available instruments.

Subsequent use of this type of instrument over a considerable period of time in several States by a district engineer of the bureau indicates that it is entirely satisfactory for the purpose. It is, therefore, with considerable confidence in the worth of the instrument that a description is now presented.

There are a number of ways in which the roughness of road surfaces may be indicated by an instrument attached to a vehicle, but the fundamental principle upon which all of them depend is that the vertical motion imparted to the vehicle by the irregularities in the road surface bears a direct relation to the degree of roughness. In order that the effect of this motion upon the chassis of the vehicle may be minimized, body springs and rubber tires are provided, the deflection of which absorbs much of the undesirable vibration. The magnitude of these deflections depends not only on the magnitude of the road roughness but also on the speed of the vehicle, amount and distribution of the load, and the type and condition of the spring and tire equipment. By maintaining constant all of these other conditions, the deflection of the body springs may be made to measure the relative roughness of the riding surfaces over which the vehicle is driven. This fact furnishes the principle on which the relative roughness determinator or roughometer, as it has been called, operates.

THE RELATIVE ROUGHNESS DETERMINATOR

Briefly, the roughometer consists of a rack which is attached in a vertical position to the front axle of the vehicle. Meshed with this rack is a spur gear which is supported by the frame of the car. Movement of the front axle with respect to the chassis, caused by deflection of the body springs, thus produces translation of the rack and rotation of the gear. This gear is connected through a flexible shaft to a mechanical counter on the instrument board of the automobile. Deflection of the front springs of the vehicle thus causes the rotation of the spindle of the counter. In order that this spindle will not rotate in the reverse direction when the body springs return from their deflected position a ball clutch is interposed between the gear mentioned and the flexible shaft which operates the counter. This ball clutch allows the flexible shaft to turn in only one direction so that the counter operates only during deflection of the body springs and thus records the summation of these deflections during the time the instrument is in operation.

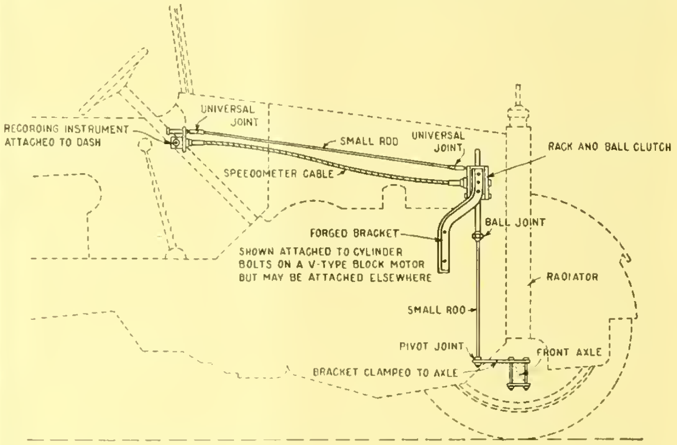


FIG. 1.—DIAGRAM OF THE RELATIVE ROUGHNESS DETERMINATOR MOUNTED ON AN AUTOMOBILE

Before proceeding to a detailed description of the instrument it may be desirable to call attention to certain of its distinctive features, which are:

1. It can be attached to any automobile without impairing the appearance or the normal operation of the vehicle.
2. The data are presented as abstract numerical factors which can be used immediately.
3. Satisfactorily accurate data can be obtained at normal highway operating speeds so that there is no interference with or from traffic.
4. The instrument can be thrown completely out of operation when not in use, thus preventing excessive wear of its moving parts.

DETAILED DESCRIPTION OF THE INSTRUMENT

Referring to Figure 1, which shows how the instrument is mounted on an automobile, it will be seen that it consists of two distinct parts; that is, the rack and ball-clutch mechanism which is mounted on the engine block or chassis of the car, and the recording mechanism which is permanently mounted on the instrument dash. Figure 2 shows these two parts complete but not mounted on the automobile.

In Figure 3, which is a detailed drawing of the entire mechanism, *A* is a plan of the instrument, looking down, with the rack and ball-clutch mechanism at the left and the recording mechanism at the right; *B* is a sectional view of *A*, the section being taken along the vertical longitudinal center line of the rack and ball-clutch mechanism, through the flexible cable and through the miter gears which drive the recording mechanism; *C* shows the end or side view of the rack and ball-clutch mechanism, and when mounted this side would be next to the radiator of the automobile; *D* is the front view of the recording mechanism as secured to the dash and as it appears to the operator; *E* is a vertical cross section through the ball clutch; *F* gives details of the latch which meshes the rack and gear and is controlled by the pull-out, throw-over handle located over the recording mechanism on the base plate of the instrument. The upper view shows the latch closed (rack and gear meshed); the center view shows the latch half open; and in the lower view the latch is open (rack and gear out of mesh, and the instrument not in operation).

The following is a detailed description of the instrument, in which the various parts are referred to by numbers as shown in Figure 3:

A metal frame (1) is supported by a suitable bracket, attached by screws (2) to the engine block or some other part of the chassis. (Fig. 1.) Through this frame passes a vertical rack (3), which is attached to the front axle through a rod with flexible couplings as shown in Figure 1. Any vertical movement of the axle with respect to the chassis thus causes this rack to move up or down.

Attached to and pivoted near the bottom of the stationary frame (1) is a movable metal frame (4) which is made in two pieces to facilitate casting and is secured at the bottom with cap screws (5) and at the top with a spacer rod (6).

At the center of the movable frame (4), carried in metal bearing boxes (7), is a hardened steel shaft (8), and through the middle of this shaft is a hole for lubrication, the oil being supplied by an oil reservoir in one bearing from oil cup (9). The steel shaft (8) has made with it a ball race which is ground to the proper dimensions after the steel is hardened.

On the shaft (8) next to the ball race is a hardened steel gear (10), and fastened to this gear by six screws surrounding the ball race is the hardened steel ball-clutch housing (11). This housing carries the steel balls (13); which are held in the housing and in position against the ball race by springs (13) and small screws (14).

The hardened steel gear (10) meshes with the vertical rack (3); consequently, any vertical movement of the rack is transmitted to the gear (10) and, by virtue of the ball-clutch housing (11) being fastened to the gear (10), the movement of the rack (3) causes rotation of the ball race or rotor and the steel shaft (8). The ball clutch will lock when the rack moves upward and release when the rack moves downward, yet, unless the shaft (8) is held so as to make the ball clutch release, it will simply rock back and forth. To prevent this friction is applied to shaft (8) by a brake wheel (15) which is held to the steel shaft by a set-screw, the braking friction being applied by a steel cable (16) that is secured around the spacer rod (6) at one end and is tightened by means of a thumb-nut (17) at the other end. Thus shaft (8) moves only in one direction, that in which the ball clutch locks.

At the end of the steel shaft (8) opposite from the oil reservoir, a flexible steel speedometer cable and housing (18) is attached. This cable transmits any movement of the steel shaft (8) to the recording mechanism. The recording mechanism or instrument is attached to the dash of the automobile within reach and in clear view of the operator. It consists of a base plate (19) and a mechanical set-back counter (20) which records the movement transmitted by the speedometer cable through a pair of miter gears (21) that are covered by a housing (22). The vertical movement of the rack (3) thus appears as a numerical factor on the mechanical counter.

At the top of the recording mechanism is a pull-out, throw-over handle (23). This handle is connected to a rod (24) fitted with a universal joint at each end, and controls the latch (25). When the handle (23) is turned 180 degrees at the recording instrument (from left to right) the latch (25) is extended, freeing gear (10) from rack (3) and putting the entire instrument out of operation. The dotted lines shown in Figure 3, *C* indicates the amount of movement necessary to do this.

Figure 4 shows the instrument completely disassembled, and in this figure attention is called to the simplicity of the various parts. Figure 5 shows the rack and ball clutch mounted under the hood of an automobile. This view shows the side opposite to that shown in Figure 2, and brings out clearly the details of the friction brake on the ball clutch.

USE OF THE INSTRUMENT IN THE FIELD

The various parts of the instrument are so designed that a movement of the rack of one inch records one unit on the counter. While it is possible to calibrate the instrument in terms of absolute surface profile, this calibration is difficult and of no practical value. The purpose of the instrument is to indicate the relative roughness of various surfaces or the changes in roughness of the same surface over a period of time. It is only necessary, therefore, to standardize the speed, load, spring, and tire conditions of the vehicle on which it is to be used and the device will be ready for service. Care should be taken to check the tire inflation frequently and obviously the speedometer on the car should be maintained in first-class condition.

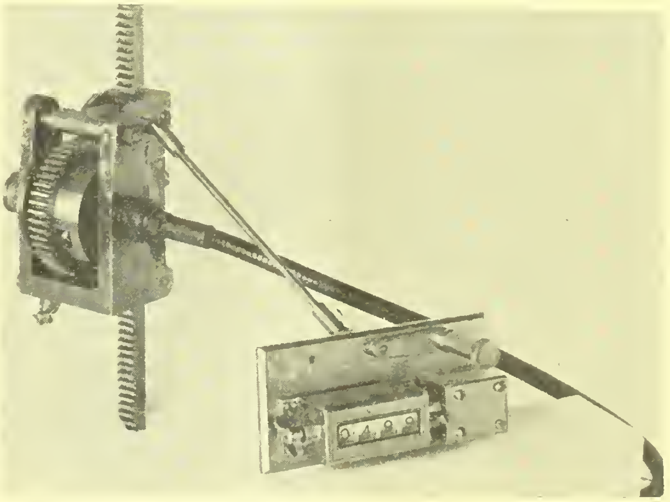


Fig. 2.—The Rack and Ball-Clutch Mechanism and the Recording Mechanism, Complete but Unmounted

Any normal operating speed may be used; but whatever the rate decided upon it should be maintained throughout the test, and the same speed should be used in comparative tests. It is desirable, therefore, to so set the speed that it can be maintained without interfering with or interference from the other traffic on the road. Table 1 gives values obtained on a section of pavement 1½ miles in length at three speeds. This table illustrates not only the effect of speed on the magnitude of the factor obtained but also the consistency of the results obtained on several runs over the same road.

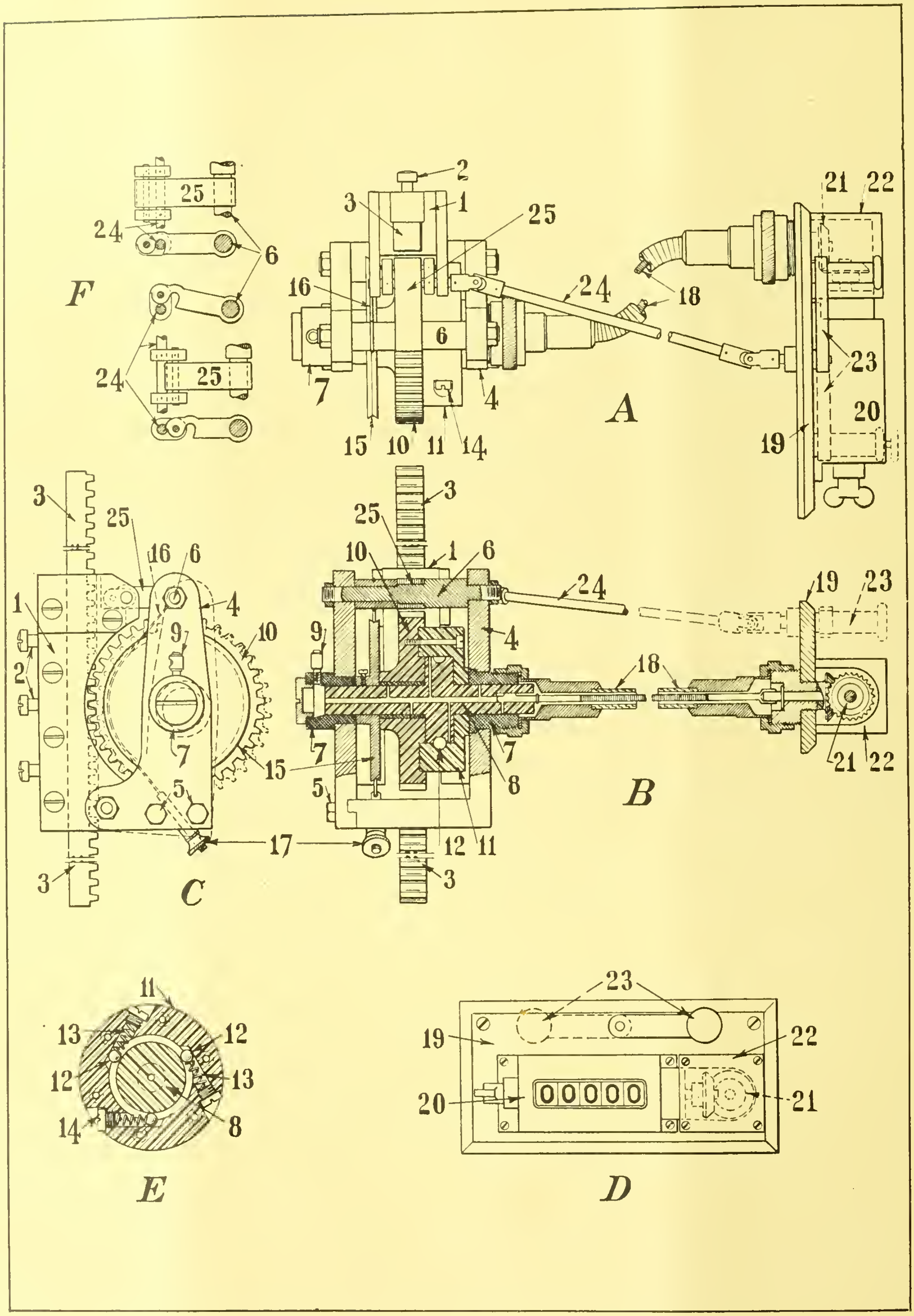


Fig. 3.—Details of the Relative Roughness Determinator

Table 1.—*Roughness factors obtained in operation over a section of pavement 1½ miles long*

THREE ROUND TRIPS AT 25 MILES PER HOUR			
Southbound	196	Northbound	173
Do	199	Do	168
Do	195	Do	170
Average	197	Average	170
FIVE ROUND TRIPS AT 30 MILES PER HOUR			
Southbound	225	Northbound	206
Do	224	Do	203
Do	227	Do	201
Do	230	Do	202
Do	225	Do	203
Average	226	Average	203
THREE ROUND TRIPS AT 35 MILES PER HOUR			
Southbound	242	Northbound	221
Do	242	Do	224
Do	246	Do	224
Average	243	Average	223

From these data it will be seen that the maximum deviation of any reading from the average of the group is considerably less than 2 per cent. It will also be noted that the increase in the roughness factor is a most directly proportional to the increase in speed.

The curves in Figure 6 indicate the relation between the speed of the vehicle and the magnitude of the roughness factor for two different vehicles equipped with different types of tire equipment. The parallel curves were made with the same car on the same length of road but running in opposite directions. One test was made near Washington, D. C., the other near Chicago, Ill.

The significance of these curves lies in the fact that they show a definite relation between the speed of the vehicle and the magnitude of the roughness factor. This relation will not be the same for different sets of equipment and should be determined for those test conditions which have been selected as standard for the equipment used.

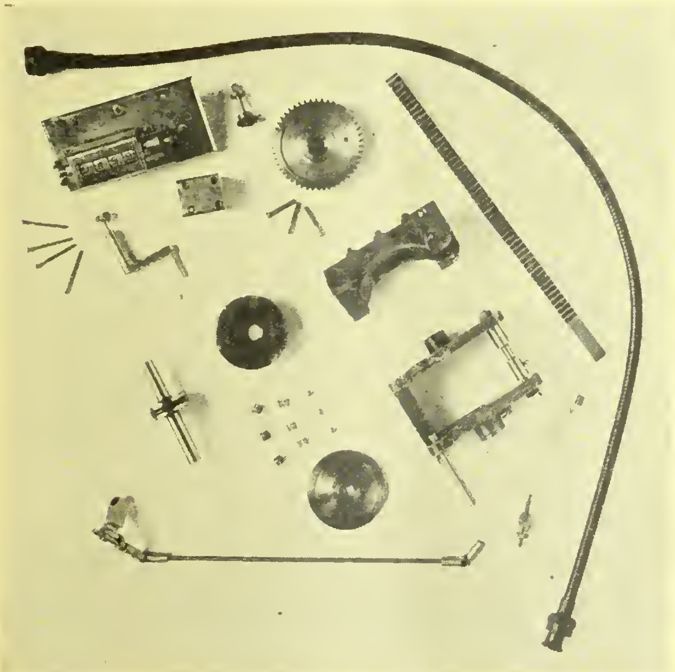


Fig. 4.—The Instrument Disassembled, Showing Simplicity of the Various Parts

Figure 7 shows graphically some data which indicate the consistency of the instrument. The car was driven over 6 miles of pavement and readings were taken every half mile. The test was repeated and although the car was driven in the same direction over exactly the same length of pavement no attempt was made to follow the same wheel path. The readings at the half-mile intervals are plotted as ordinates on the chart. Although there is a wide variation in the roughness of the different half-mile lengths, the readings on the instrument check very well at the various points. The total readings, i. e., the sum of the readings for the 12 half-mile lengths as obtained on the two test runs, check within 2 per cent. These data are representative and were selected so as to present a fair example of the consistency of the instrument.

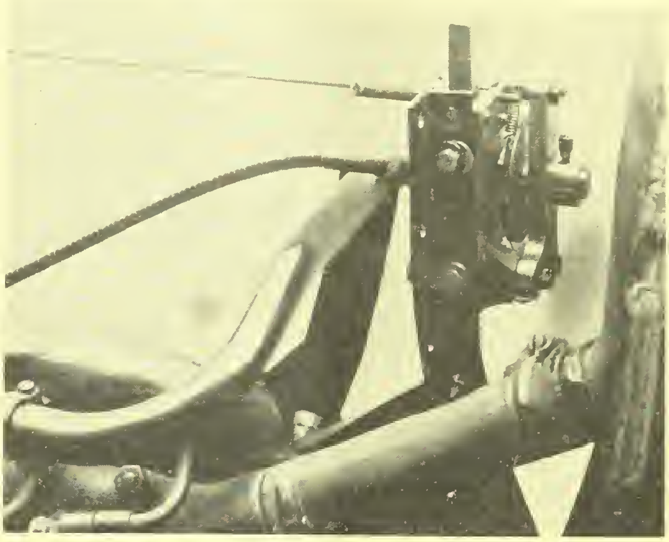


Fig. 5.—The Rack and Ball Clutch Mounted on an Automobile

The effect of the load in the car or of the type and condition of the body springs is of considerably less importance but should receive attention as there is no doubt that they do affect the results obtained. Tire size, type, and inflation pressure are very important and every effort should be made to keep these constant on all tests the results of which are to be compared.

INTERPRETATION OF THE ROUGHNESS FACTOR

The relative roughness of a road surface is indicated by an abstract number which is a measure of the total average accumulated compression of the two front springs of the vehicle in inches per mile. If the counter be set at zero at the beginning of a trip and the instrument is thrown into operation while the automobile traverses 1 mile of road surface, then the reading on the counter as the car completes this 1 mile of travel may be called the roughness factor for that particular mile of surface. Some idea of what a roughness factor of 100, 200, or 300, means may be gained from the observations following which are based on a heavy car driven at a rate of from 30 to 40 miles per hour.

Roughness factor:	Motion of vehicle
Less than 80	No perceptible vibration.
80 to 110	Slight vibration.
110 to 150	Noticeable vibration and slight lurching.
150 to 200	Disagreeable vibration and noticeable lurching.
200 to 300	Disagreeable lurching and somewhat dangerous for light cars at high speeds.

The magnitude of the roughness factor obtained on various types and conditions of road surfaces has been shown to vary somewhat with the auxiliary equipment used with the roughometer.

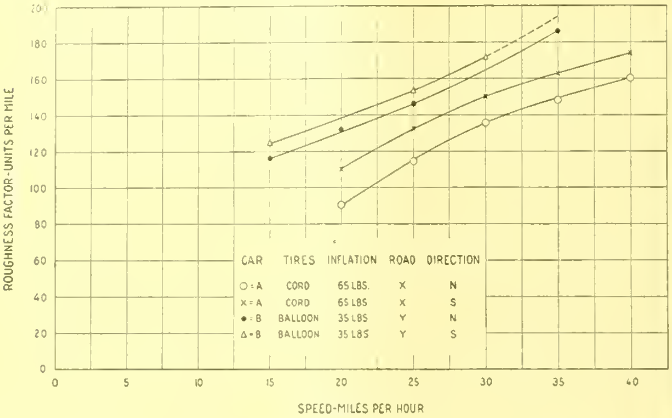


Fig. 6.—Effect of Vehicle Speed on the Roughness Factor as Determined With Two Different Cars, One Equipped With Cord Tires and One Equipped with Balloon Tires

Measurements made in the vicinity of Washington, D. C., indicated a range of from 80 to 100 for very smooth pavements to from 250 to 300 for rather rough surfaces, while some results recently obtained in California with another instrument on another type of automobile range from about 60 for good concrete pavement to 250 for a rather rough oiled gravel.

ROUGHNESS OF DIFFERENT ROADS COMPARED WITH ROUGHOMETER

A brief discussion of some of the data obtained last year by one of the district engineers of the bureau may be of interest. Forty-eight sections of concrete pavement, comprising 412 miles, and five sections of bituminous topped pavement, comprising 59 miles, were measured. The average roughness factors found for particular sections of the two types were, respectively, 141 and 193. The average roughness factor for concrete pavements constructed prior to 1923 is 196, whereas that for those constructed during and since 1923 is 115, showing that very great progress has been made toward constructing smoother pavements.

Data on a section of pavement fairly typical of those constructed six or seven years ago are shown in Table 2, and Table 3 shows similar data for a section constructed in 1924.

Table 2.—*Roughness factors of 5 miles of concrete pavement typical of those constructed prior to 1920*

Mile of pavement	Roughness factor	Mile of pavement	Roughness factor
First	252	Fourth	229
Second	258	Fifth	349
Third	244	Average for 5 miles	266

Table 3.—*Roughness factors of 13 miles of concrete pavement constructed in 1924*

Mile of pavement	Roughness factor	Mile of pavement	Roughness factor
First	90	Ninth	85
Second	90	Tenth	82
Third	81	Eleventh	81
Fourth	84	Twelfth	(hills) 102
Fifth	(hills) 102	Thirteenth	83
Sixth	98		
Seventh	99	Average for 13 miles	89.3
Eighth	84		

In each of three States it was noticed that the first mile constructed was generally the roughest mile in the section. The average roughness factor for the roughest mile in 48 sections in one State was 175 and of the smoothest mile 107, a difference of 68. This difference for 34 sections in another State was 65, and for 13 sections in a third State was 67. This would seem to indicate that very careful supervision should be exercised during the construction of the first mile of pavement laid and that as many first miles should be avoided as possible by awarding contracts early in the season and of such length as may require the entire season to construct. This well illustrates the importance of the roughometer or relative roughness determinator to the highway engineer.

Table 4 presents data which show the greatly increased roughness in the first mile of construction.

Table 4.—*Roughness factors, by miles, on a 7-mile section of pavement, showing increased roughness in the first mile*

Mile of pavement	Roughness factor	Mile of pavement	Roughness factor
First	383	Fifth	115
Second	271	Sixth	126
Third	128	Seventh	156
Fourth	112		

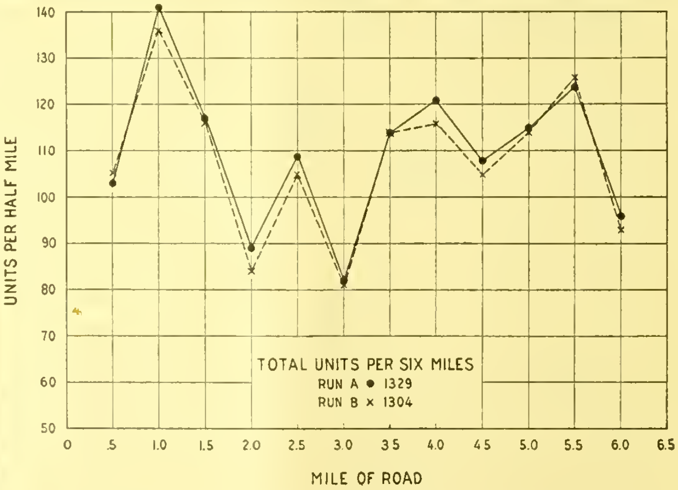


Fig. 7.—Results of Test of Consistency of Instrument in Which Two Runs Were Made in the Same Direction Over the Same Road at a Speed of 20 Miles per Hour. No Attempt was Made to Follow the Same Wheel Path

In general, it has been found that a pavement is smooth or rough all the way across, although in a number of instances a consistent difference between the right and left sides has been observed. Table 5 presents data typical of such a case.

(Continued on p. 151)

THE FLOW OF WATER THROUGH CULVERTS

CONCLUSIONS FROM REPORT ON TESTS AT UNIVERSITY OF IOWA

THE Bureau of Public Roads and the State University of Iowa have recently made a large number of tests of the flow of water through short conduits, such as pipe and box culverts and sluiceways under levees. These tests were conducted at the hydraulic laboratory of the university and have been reported on in Bulletin 1, University of Iowa Studies in Engineering, "The flow of water through culverts," by D. L. Yarnell, drainage engineer, United States Department of Agriculture, in collaboration with Floyd A. Nagler, associate professor of mechanics and hydraulics, and Sherman M. Woodward, professor of mechanics and hydraulics, both of the University of Iowa.

The investigations were undertaken primarily for the purpose of determining—

1. The quantity of water that will flow through culverts or sluiceways under levees of different materials, sizes, and shapes under conditions of actual use.
2. What conditions tend to increase or decrease such quantity.
3. What principles should be followed in design to secure the greatest discharging capacity for the least cost.

The report describes the methods of making the tests and presents the experimental results together with the discharge formulas developed for the various culverts.

The following conclusions are drawn from the results of 1,480 experiments on pipe culverts made of concrete, vitrified clay, and corrugated metal of the following sizes: 12, 18, 24, and 30 inches in diameter; and 1,821 tests on concrete box culverts of the following sizes: 2 by 2 feet, 3 by 3 feet, 4 by 4 feet, 4 by 3 feet, 4 by 2¼ feet, 4 by 2 feet, 4 by 1 foot, and 4 by one-half foot. Tests were made on the culverts flowing partly full and full, both with a free and submerged outlet. Experiments were also run with various types of entrances.

1. The discharging capacity of a culvert depends primarily upon the cross section of the culvert and the difference in water level at the two ends of the culvert.

2. To obtain the maximum discharge the culvert must be so laid as to insure the full cross section of the culvert being filled by the flowing water.

3. If the culvert is laid at too high an elevation with respect to the water levels at the two ends, it will not run full and hence will not attain its maximum capacity.

4. If a culvert is so laid that both its upstream and downstream ends are completely submerged, the amount of water which it discharges will be proportional to the square root of the difference in water level at the two ends; and the exact grade at which the culvert is laid has no effect whatever upon its maximum discharging capacity.

5. The difference in water level at the two ends, called hereafter the head on the culvert, is utilized in three ways—first, in overcoming friction around the entrance corner; second, in overcoming friction along the walls throughout the barrel of the culvert; third, in imparting the velocity necessary to the water in entering the culvert. The three portions into which the total head is thus divided are called for convenience entrance loss, friction loss, and velocity head, respectively.

The following general conclusions numbered from 6 to 25, inclusive, are drawn from the results of the tests on the pipe culverts:

6. The coefficient of roughness, *n*, in the Kutter formula for the concrete pipe ranges from 0.012 for the 12-inch size to 0.013 for the 30-inch size.

7. The coefficient of roughness, *n*, in the Kutter formula, for the vitrified-clay pipe ranges from 0.010 for the 12-inch size to 0.013 for the 30-inch size.

8. The coefficient of roughness, *n*, in the Kutter formula, for the corrugated-metal pipe ranges from 0.019 for the 12-inch size to 0.023 for the 30-inch size.

9. In concrete, vitrified-clay, and corrugated-metal pipe culverts, 30.6 feet long, with straight end-wall entrances:

(*a*) The 12-inch concrete pipe with beveled lip end upstream discharges about 49 per cent more water than the 12-inch metal pipe.

(*b*) The 18-inch concrete pipe with beveled lip end upstream discharges about 40 per cent more water than the 18-inch metal pipe.

(*c*) The 24-inch concrete pipe with beveled lip end upstream discharges about 36 per cent more water than the 24-inch metal pipe.

(*d*) The 30-inch concrete pipe with beveled lip end upstream discharges about 32 per cent more water than the 30-inch metal pipe.

(*e*) The 12-inch clay pipe discharges about 65 per cent more water than the 12-inch metal pipe.

(*f*) The 18-inch clay pipe discharges about 50 per cent more water than the 18-inch metal pipe.

(*g*) The 24-inch clay pipe discharges about 40 per cent more water than the 24-inch metal pipe.

(*h*) The 30-inch clay pipe discharges about 30 per cent more water than the 30-inch metal pipe.

The relative capacities of these culverts may also be expressed in the following terms which are mathematically equivalent to the above.

(*i*) The 12-inch metal pipe has about 67 per cent of the carrying capacity of the 12-inch concrete pipe with beveled lip end upstream.

(*j*) The 18-inch metal pipe has about 71 per cent of the carrying capacity of the 18-inch concrete pipe with beveled lip end upstream.

(*k*) The 24-inch metal pipe has about 74 per cent of the carrying capacity of the 24-inch concrete pipe with beveled lip end upstream.

(*l*) The 30-inch metal pipe has about 76 per cent of the carrying capacity of the 30-inch concrete pipe with beveled lip end upstream.

(*m*) The 12-inch metal pipe has about 61 per cent of the carrying capacity of the 12-inch clay pipe.

(*n*) The 18-inch metal pipe has about 68 per cent of the carrying capacity of the 18-inch clay pipe.

(*o*) The 24-inch metal pipe has about 73 per cent of the carrying capacity of the 24-inch clay pipe.

(*p*) The 30-inch metal pipe has about 78 per cent of the carrying capacity of the 30-inch clay pipe.

10. In concrete pipe culverts, 30.6 feet long, with straight end-wall entrances:

(*a*) The 12-inch pipe with beveled lip end upstream discharges about 5 per cent more water than the same pipe with a square-cornered entrance.

(*b*) The 18-inch pipe with beveled lip end upstream discharges about 9 per cent more water than the same pipe with a square-cornered entrance.

(*c*) The 24-inch pipe with beveled lip end upstream discharges about 12 per cent more water than the same pipe with a square-cornered entrance.

(*d*) The 30-inch pipe with beveled lip end upstream discharges about 14 per cent more water than the same pipe with a square-cornered entrance.

11. Due to the larger amount of pipe friction in corrugated-metal pipes, a change in culvert length produces a greater change in discharge than with concrete and vitrified-clay pipe culverts.

12. The 45-degree wing walls used in connection with a corrugated-metal pipe culvert increase the capacity from 1 to 10 per cent over that obtained in a metal pipe culvert with a straight end wall.

13. The 45-degree wing walls used in connection with a corrugated-metal pipe culvert are more efficient when set flush with the edge of the pipe than when set 6 inches back from the edge of the pipe.

14. The 45-degree wing walls used in connection with a corrugated-metal pipe culvert are more efficient when built full height to the top of the head wall than when constructed only to the standard height shown in Figure 6.[1]

15. The 45-degree wing walls used in connection with a vitrified-clay pipe culvert produce a carrying capacity substantially equal to that with the regular bell end upstream.

16. The U-type wing walls used in connection with a vitrified-clay pipe culvert produce a carrying capacity slightly less than that with the straight end wall.

17. The beveled lip end at the entrance of a concrete pipe culvert is a great aid in reducing the entrance loss, especially in the larger sizes.

18. The bell end at the entrance of a vitrified-clay pipe culvert by virtue of its shape greatly reduces the entrance loss below that produced by a right-angle corner, especially in the smaller sizes.

19. A 24-inch clay pipe, 38 feet long, with a straight end wall and the regular bell end upstream carries about 10 per cent more water than the same culvert with a square-cornered entrance.

20. Merely rounding the entrance to a 24-inch vitrified-clay pipe culvert increases the capacity approximately 13 per cent over that obtained with a square-cornered entrance.

21. By projecting the pipe through the head wall so as to obtain the effect on the discharge of no head wall, it was found that:

(*a*) In the 12-inch concrete pipe culvert having a square cornered entrance, there was little difference whether the pipe projected 3 inches, 2 feet, or 4 feet beyond the head wall.

(*b*) The discharge is decreased slightly by projecting the pipe through the head wall as compared with the same culvert with a straight end wall entrance.

(*c*) The 18-inch corrugated-metal pipe culvert with a 3-inch projection beyond the head wall carries slightly more water than the same pipe with either a 2-foot or a 4-foot projection.

(*d*) The 18-inch metal pipe culvert with a straight end wall entrance carries more water than the same pipe with any length of projection.

22. By doubling the area of the outlet end of an 18-inch vitrified-clay pipe culvert by attaching a conical section, the sides of which diverge at an angle of about 10 degrees, the discharge of the culvert, when the outlet end is submerged, is increased about 40 per cent over that obtained through the same culvert having a uniform bore throughout.

23. In the formulas for discharge the average exponent of H, the head on the pipe culvert, for Tables 22 to 79,[1] inclusive, is 0.488. In hydraulics the general practice is to assume that the discharge varies as the square root of the head.

24. New discharge formulas for the flow of water through concrete, vitrified-clay, and corrugated-metal culvert pipe have been derived from the experimental data in this report. The formulas as derived for culverts 30.6 feet long with straight end wall entrances are as follows:

Concrete pipe with beveled lip end upstream:

$$Q=4.61\ D^{2.18}\ H^{0.50} \tag{1}$$

Concrete pipe with square cornered entrance:

$$Q=4.40\ D^{2.09}\ H^{0.50} \tag{2}$$

Vitrified-clay pipe with bell end upstream:

$$Q=5.07\ D^{2.05}\ H^{0.50} \tag{3}$$

Corrugated-metal pipe:

$$Q=3.10\ D^{2.31}\ H^{0.50} \tag{4}$$

25. The general discharge formulas derived for pipe culverts with straight end-wall entrance and of any size and length, when flowing full, are as follows:

Concrete pipe, beveled lip entrance:

$$Q=\frac{A\sqrt{2g\ H}}{\sqrt{1.1+\dfrac{0.026\ L}{D^{1.2}}}} \tag{13}$$

Concrete pipe, square-cornered entrance:

$$Q=\frac{A\ \sqrt{2g\ H}}{\sqrt{1+0.31\ D^{0.5}+\dfrac{0.026\ L}{D^{1.2}}}} \tag{14}$$

Vitrified-clay pipe, regular bell end upstream:

$$Q=\frac{A\ \sqrt{2g\ H}}{\sqrt{1+0.023\ D^{1.9}+\dfrac{0.022\ L}{D^{1.0}}}} \tag{15}$$

Corrugated-metal pipe:

$$Q=\frac{A\ \sqrt{2g\ H}}{\sqrt{1+0.16\ D^{0.6}+\dfrac{0.106\ L}{D^{1.2}}}} \tag{16}$$

In these formulas—

Q=discharge in cubic feet per second.
A=cross-sectional area of pipe in square feet.
D=diameter of pipe in feet.
L=length of culvert in feet.
H=head on pipe in feet or the difference in the water level at the two ends of the culvert.
g=acceleration of gravity.

The following general conclusions, numbered from 26 to 33, are drawn from the results of the tests on the concrete box culverts:

26. Concrete box culverts with straight head walls and rounded lip entrance discharge from 8 to 12 per cent more water than the same size culvert with square-cornered entrance in the sizes tested in this investigation.

27. Concrete box culverts with straight head walls and beveled lip entrance discharge from 7 to 9 per cent more water than the same size culvert with square-cornered entrance in the sizes tested in this investigation.

28. If the outlet end of a 36-foot box culvert with a rounded lip entrance is flared by diverging the sides at an angle of 6° 30′ throughout a distance of 10 to 12 feet from the outlet head wall, thus doubling the area of its cross section at the outlet, the capacity of the culvert is increased about 60 per cent above the capacity of a similar culvert 36 feet long with the uniform bore extending the entire length of the culvert.

29. The 2 by 2 foot box culvert, 30 feet long, with a rounded lip entrance and flared on the two sides for its entire length to a 4 by 2 foot opening at the outlet end will discharge 86 per cent more water than a 2 by 2 by 30 foot box culvert of uniform bore with a square-cornered entrance, when both culverts are flowing full.

30. The 2 by 2 foot box culvert, 30 feet long, with a rounded lip entrance and flared on the two sides for its entire length to a 4 by 2 foot opening at the outlet end will discharge the same quantity of water as a 3.61 by 2 by 30 foot box culvert of uniform bore with a square-cornered entrance, when both culverts are flowing full.

31. The following general discharge equations have been developed for concrete box culverts with straight end wall entrances when flowing full:

Box culverts with rounded lip entrances:

$$Q=\frac{A\ \sqrt{2g\ H}}{\sqrt{1.05+\dfrac{0.0045\ L}{R^{1.25}}}} \tag{17}$$

Box culverts with square-cornered entrances:

$$Q=\frac{A\ \sqrt{2g\ H}}{\sqrt{1+0.4\ R^{0.3}+\dfrac{0.0045\ L}{R^{1.25}}}} \tag{18}$$

in which the same symbols are used as in the pipe culvert formulas and the term, R, is the mean hydraulic radius of the culvert in feet.

32. Rectangular concrete box culverts require more head to overcome friction than square concrete box culverts of the same area, and hence have a smaller carrying capacity provided the entrance losses (in head) are the same. The head lost in friction for culverts of the same area varies inversely with the hydraulic radius. The entrance losses, however, on the rectangular culverts tested are less than those on the square culverts of the same area and type of entrance. Since one culvert differs from another in capacity per square foot of area inversely in proportion to the square root of the sum of all head losses involved, it is possible for a rectangular culvert to have a slightly greater capacity than a square culvert, although the reverse is usually the case.

33. The chamfering of the corners of box culverts reduces the discharge of the culvert by an insignificant amount, an amount which is less in per cent than the corresponding reduction in culvert area produced by the chamfering.

A comparison of the results of the tests on the concrete box culverts with the results of the tests on the pipe culverts reveals the following facts:

34. A 24-inch concrete pipe culvert, 30 feet long, with beveled lip end upstream, carries 19 per cent less water than a 2 by

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[1] Not reproduced in this summary.

2 by 30 foot concrete box culvert with square-cornered entrance (when the two culverts are flowing full), but the concrete pipe culvert carries about 7 per cent more water per square foot of waterway than the box culvert.

35. A 24-inch corrugated-metal pipe culvert, 30 feet long, carries 57 per cent less water than a 2 by 2 by 30 foot box culvert with square-cornered entrance (when the two culverts are flowing full), and the metal pipe carries about 23 per cent less water per square foot of waterway than the box culvert.

36. A 27-inch vitrified-clay pipe culvert 30 feet long (containing approximately the same cross-sectional area as a 2 by 2 foot box culvert) will carry 7 per cent more water than the 2 by 2 by 30 foot box culvert with square-cornered entrance when both culverts are flowing full.

37. The value of the entrance loss coefficient for a given type of entrance corner varies with the shape of the cross section of the culvert. That cross section having the greatest hydraulic radius for a given culvert area has also the greatest entrance loss coefficient. That is, the entrance loss coefficient is greater for a circular culvert than for one with a square section, and the coefficient for the square section is greater than that for the rectangular section.

38. The results on the various types of entrances and outlets have an important bearing on the design of suction and discharge pipes for drainage and irrigation pumping plants. Sluiceways under levees are hydraulically like road culverts. The data in this report will likewise be found directly applicable to many irrigation structures.

Copies of the bulletin may be obtained from the university editor at $1 a copy.

(Continued from p. 148)

TABLE 5.—*Roughness factors on east and west sides of a pavement*

Mile of pavement	Roughness factor		Mile of pavement	Roughness factor	
	West side	East side		West side	East side
First	81	89	Ninth	89	97
Second	91	97	Tenth	82	81
Third	88	89	Eleventh	83	82
Fourth	101	121	Twelfth	85	88
Fifth	69	77	Thirteenth	79	81
Sixth	76	86	Fourteenth	94	85
Seventh	74	84			
Eighth	85	96	Average per mile	84	89.5

As indicative of how smooth pavements can be built, Table 6 shows data obtained for the smoothest pavements found in two Midwestern States.

TABLE 6.—*Roughness factors on smoothest pavements found in two States*

	Roughness factor		Roughness factor
(*a*) Mile of pavement:		Second	67
First	63	Third	60
Second	71	Fourth	63
Third	66	Fifth	57
Fourth	119	Sixth	63
Fifth	109	Seventh	59
		Eighth	62
Average for first 3 miles	66.7	Ninth	59
		Tenth	68
(*b*) Mile of pavement:			
First	60	Average	61.8

From data similar to that cited in Table 6, obtained on many hundreds of miles of roads the indications are that the instrument as described does give a true index of the relative roughness of road surfaces.

The data, which can be obtained quickly and at small cost, should be invaluable to highway officials, particularly supervisory officials who can not hope to inspect every mile of pavement, as a basis for rating contractors, superintendents on force account work, engineers, inspectors, and all others directly responsible for the quality of the pavement. They will be found valuable even by the highway official who can and does visit all projects during construction for they will furnish him definite data rather than opinions on which to base conclusions.

INTERNATIONAL ROAD CONGRESS WILL ACCEPT INVITATION TO MEET IN UNITED STATES IN 1929

The International Road Congress will accept an invitation to meet in the United States in 1929, according to Senator Albert Mathieu, of France, president of the association. This assurance was given at the close of the congress held at Milan, Italy, September 6 to 14 and in response to the statement by Thomas H. MacDonald, Chief of the United States Bureau of Public Roads and chairman of the official delegation from the United States, that the United States Congress would be urged to extend such an invitation.

The congress at Milan was attended by 2,000 delegates from 50 countries and 5 continents, and the United States was officially represented for the first time. Mr. MacDonald was made a member of the council of the congress.

In the discussions of the congress, the United States and Great Britain declined to adhere to a resolution commending as a general policy the private operation of government subsidized toll roads, which has been tried out successfully on some stretches of Italian road. The United States delegates advanced the argument that roads are a government function and should be open to the use of all except in rare cases where private construction should eventually result in public operation. A resolution providing for all viewpoints was adopted.

Following the meeting in Milan the United States delegates went to Rome, where they were received in a private audience by Premier Mussolini, who asked them to convey a message to the United States that they had seen a disciplined and tenacious people courageously at work and desiring only peace. In response to Mr. MacDonald's statement that in modern highway transportation there is a new way of living ahead for the masses, the premier stated his conviction that motor roads must be built everywhere as railroads and carriage roads have been built in the past.

The American delegation left Rome for a survey of traffic and road conditions in western Europe and will sail from Southampton on October 12.

MOTOR VEHICLE REGISTRATIONS FOR THE FIRST SIX MONTHS OF 1926[1]

States	Registered motor vehicles individually and commercially owned [2]			Special list of passenger cars for hire [3]		Other registered vehicles [4]		Tax-exempt official motor vehicles and motor cycles [5]			Number of licenses or permits (automobile)			Grand total registered motor cars and trucks first six months of 1925	Per cent increase 1926 over 1925	States
	Grand total registered motor cars and trucks 1926	Passenger automobiles, taxis, and busses [6]	Motor trucks and road tractors [6]	2 to 7 passenger capacity	Over 7 passenger capacity	Trailers	Motor cycles	United States cars [7]	State and local cars	Motor cycles (official)	Dealers	Operators	Chauffeurs			
Alabama	197,602	173,383	24,219	2,163	76	740	313	167	----------	----------	387	----------	1,604	182,561	8.2	Alabama.
Arizona	64,165	56,312	7,853	371	95	----------	295	176	656	----------	212	----------	104	59,809	7.3	Arizona.
Arkansas	177,235	152,537	24,698	1,917	167	1,150	193	39	542	34	472	----------	4,141	148,981	19.0	Arkansas.
California	1,459,570	1,262,841	[6] 196,729	4,765	1,842	26,353	8,968	1,217	18,716	360	2,939	----------	88,355	1,317,825	10.8	California.
Colorado	225,810	208,648	17,162	1,361	426	----------	1,168	283	----------	----------	3,128	----------	----------	213,891	5.6	Colorado.
Connecticut	238,727	201,253	37,474	1,874	465	256	2,718	71	1,059	168	349	255,719	3,205	216,746	10.1	Connecticut.
Delaware	40,303	32,625	[6] 7,678	190	49	150	360	44	----------	----------	595	40,875	3,625	35,600	13.2	Delaware.
Florida	407,777	343,349	[6] 64,428	2,423	522	----------	1,146	75	2,265	161	3,106	----------	8,848	231,439	76.2	Florida.
Georgia	238,618	208,602	30,016	1,664	161	----------	678	934	----------	----------	806	----------	2,856	207,663	14.9	Georgia.
Idaho	84,161	77,276	6,885	359	57	153	411	103	1,077	21	386	----------	303	73,500	14.5	Idaho.
Illinois	1,217,265	1,062,844	154,421	10,033	1,250	2,537	4,627	979	----------	----------	4,392	----------	80,856	1,123,084	8.4	Illinois.
Indiana	690,704	598,338	92,366	3,648	1,177	4,521	2,865	3,184	3,308	----------	2,307	----------	32,400	659,782	4.7	Indiana.
Iowa	648,282	601,011	47,271	1,999	259	----------	1,739	44	2,500	79	2,205	----------	9,462	611,002	6.1	Iowa.
Kansas	433,561	390,951	[6] 42,610	1,690	274	----------	988	192	2,084	51	2,371	----------	----------	406,990	6.5	Kansas.
Kentucky	247,104	222,487	24,617	2,918	347	----------	517	90	1,300	49	1,098	----------	7,616	233,828	5.7	Kentucky.
Louisiana	216,500	183,000	33,500	970	261	----------	390	209	----------	----------	----------	----------	----------	190,896	13.4	Louisiana.
Maine	128,466	108,625	19,841	1,977	186	604	801	64	992	68	1,140	150,027	5,104	126,200	1.8	Maine.
Maryland	227,491	216,634	10,857	2,266	581	518	3,255	1,969	----------	----------	5,441	18,268	12,813	208,338	9.2	Maryland.
Massachusetts	627,736	537,761	89,975	6,002	1,354	436	6,746	556	----------	----------	2,019	404,370	----------	576,103	9.0	Massachusetts.
Michigan	992,178	862,916	[6] 129,262	3,153	1,703	10,093	2,835	371	5,975	----------	2,025	110,041	63,009	798,460	24.3	Michigan.
Minnesota	574,356	512,163	62,193	1,616	605	2,161	2,326	252	213	----------	2,103	----------	----------	519,168	10.6	Minnesota.
Mississippi	180,030	161,519	18,511	1,114	144	3,000	68	74	----------	----------	5,087	----------	----------	148,758	21.0	Mississippi.
Missouri	583,450	525,333	58,117	4,528	664	1,402	1,501	311	1,305	13	2,159	4,435	20,855	535,528	8.9	Missouri.
Montana	92,340	79,608	[6] 12,732	526	117	----------	157	229	971	----------	423	----------	316	83,950	10.0	Montana.
Nebraska	329,669	304,922	[6] 24,747	1,150	87	926	926	226	----------	----------	2,409	----------	----------	295,341	11.6	Nebraska.
Nevada	20,527	16,480	4,047	119	12	39	66	42	373	6	110	----------	----------	17,939	14.4	Nevada.
New Hampshire	78,979	70,291	8,688	1,757	144	436	1,107	22	650	----------	474	56,000	30,000	73,120	8.0	New Hampshire.
New Jersey	575,237	467,385	107,852	5,158	2,807	1,365	5,819	708	5,031	775	2,501	654,305	----------	505,474	13.8	New Jersey.
New Mexico	46,571	44,894	1,677	346	54	107	151	156	----------	4	130	----------	----------	42,205	10.3	New Mexico.
New York	1,562,492	1,302,359	260,133	31,892	3,113	4,964	15,152	1,666	10,847	----------	4,221	1,182,902	393,050	1,404,653	11.2	New York.
North Carolina [1]	371,353	338,192	33,161	1,871	254	500	1,057	429	4,443	----------	761	----------	----------	[1] 315,000	10.8	North Carolina.
North Dakota	144,079	134,331	9,748	292	28	----------	241	3	122	----------	----------	----------	----------	126,106	14.3	North Dakota.
Ohio	1,370,756	1,219,202	151,554	4,811	1,717	8,784	7,117	2,362	7,150	258	3,652	----------	3,756	1,292,000	6.1	Ohio.
Oklahoma	490,000	445,000	45,000	1,886	271	----------	732	530	----------	----------	2,300	----------	----------	420,000	16.7	Oklahoma.
Oregon	195,641	181,211	[10] 14,430	686	357	([10])	1,788	141	----------	----------	561	22,845	10,946	179,566	9.0	Oregon.
Pennsylvania	1,326,682	1,143,932	182,750	6,455	1,837	2,852	11,003	1,383	3,018	867	26,216	1,372,753	----------	1,205,287	10.1	Pennsylvania.
Rhode Island	96,652	80,163	16,489	1,282	191	57	1,014	56	540	68	263	64,540	----------	89,247	8.3	Rhode Island.
South Carolina	151,012	137,397	13,615	1,060	80	778	142	91	----------	----------	480	----------	77	141,208	6.9	South Carolina.
South Dakota	155,763	142,680	13,083	319	57	----------	192	85	802	----------	1,027	----------	----------	150,335	3.6	South Dakota.
Tennessee	227,775	207,894	19,881	1,972	219	----------	503	132	2,066	60	506	----------	----------	218,735	4.1	Tennessee.
Texas	904,050	820,476	83,574	5,130	531	4,745	1,980	2,505	----------	----------	2,877	----------	8,466	848,661	6.5	Texas.
Utah	81,830	70,877	10,953	322	94	402	482	173	705	----------	281	----------	2,269	76,410	7.1	Utah.
Vermont	62,899	58,168	4,731	715	102	81	464	28	----------	----------	341	56,532	12,428	60,424	4.1	Vermont.
Virginia	277,125	241,500	35,625	1,816	548	420	1,599	1,141	2,551	226	3,605	----------	6,704	261,400	6.0	Virginia.
Washington	326,500	280,776	45,724	1,044	483	1,341	2,106	637	4,028	142	1,200	----------	----------	293,589	11.2	Washington.
West Virginia	183,788	162,530	21,258	1,276	153	245	962	33	1,739	----------	11,127	35,390	21,050	184,200	([11])	West Virginia.
Wisconsin	590,797	520,134	70,663	2,568	591	----------	2,433	92	300	63	2,693	----------	----------	534,662	10.5	Wisconsin.
Wyoming	44,367	39,608	4,759	286	321	----------	149	209	205	----------	275	----------	----------	42,600	4.1	Wyoming.
District of Columbia	89,857	78,356	11,501	995	247	----------	1,186	837	1,801	181	1,794	5,973	----------	82,427	9.0	District of Columbia.
Grand total	19,697,832	17,288,774	2,409,058	134,735	27,080	82,116	103,436	[12] 33,179	89,334	3,654	114,954	4,434,975	834,218	[8] 17,770,691	10.8	Grand total.

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[1] All States but North Carolina report details for first six months, 1926. North Carolina has registration year ending June 30, which full year is reported here.

[2] The first three columns record the regularly registered motor cars and trucks excluding where possible nonresident registrations and reregistrations. The grand total registration in the first column is subdivided into passenger-carrying cars shown in the second column, and motor trucks and road tractors shown in the third column.

[3] Special list showing passenger cars for hire on which there was paid internal revenue tax during fiscal year ending June 30, 1926. These vehicles are included in the grand total as a result of their regular registration.

[4] Shows the registered trailers; some States do not register trailers, or they are included with motor trucks. The registered motor cycles with or without side cars are shown. Both classes here listed are not included in grand total.

[5] Shows motor vehicles and motor cycles which are tax-exempt and owned by United States Government, States, counties, and municipalities, etc. These vehicles are not included in grand total registration.

[6] Busses reported with trucks in several States as noted in third column.

[7] Data obtained from United States Government sources as of June 30, 1925.

[8] Corrected figure—previously published figure included reregistrations.

[9] Includes about 7,800 cars owned by public-service corporations, not taxed by law.

[10] Oregon includes trailers with motor trucks

[11] Slight decrease due to delay in registrations caused by new law on title certificates.

[12] Includes 7,859 "cars at large" not allocated to any State.

RECEIPTS FROM MOTOR VEHICLE FEES, ETC., FOR THE FIRST SIX MONTHS OF 1926[1]

States[2]	Total gross receipts	Subdivision of registration receipts[1]					Miscellaneous receipts[1]			Disposition of gross receipts					States
		Motor cars			Other vehicles						For highway purposes				
		Total motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor cycles	Dealer's licenses	Chauffeur and operator permits	Miscellaneous	Collection and administration	State highways	Local roads	State and county road bonds[9]	For other purposes	
Alabama	$2,638,532									[3] $92,265	$2,031,753	$514,514			Alabama.
Arizona	389,620										389,620				Arizona.
Arkansas*	3,793,915	$3,668,825	$3,074,965	$593,860	$17,250	$1,995	$47,200	$20,705	$37,940	13,000	1,780,915	802,198	c$1,197,802		Arkansas.
California*	7,163,610	6,677,047	4,139,074	2,537,973	221,393	31,650	28,811	181,974	22,735	1,432,722	2,865,444	2,865,444			California.
Colorado*	1,389,008	1,315,924	1,107,921	208,003		2,336	70,748			69,450		659,779	659,779		Colorado,
Connecticut*	5,587,387	4,473,897	3,446,691	1,027,206	7,306	11,091	17,241	783,182	294,670		5,587,387				Connecticut.
Delaware*	702,739	544,914	392,864	152,050	2,512	1,440	6,820	133,203	13,850		702,739				Delaware.
Florida	5,513,054									275,653	3,859,138	1,378,263			Florida.
Georgia*	3,101,444	3,048,980	2,543,575	505,405		3,383	40,300	5,709	3,072	84,503	3,016,941				Georgia.
Idaho*	1,266,811	1,235,648	1,049,513	186,135	3,745	2,055	21,940	606	2,817		114,013	1,140,130	12,668		Idaho.
Illinois*	13,049,809	12,357,343	9,416,020	2,941,323	53,285	17,468	87,802	284,065	249,846		7,539,809		5,510,000		Illinois.
Indiana*	4,672,329	4,427,564	3,448,325	979,239	24,378	5,724	46,100	32,453	136,110		4,672,329				Indiana.
Iowa	9,672,121									338,524	9,333,597				Iowa.
Kansas	5,202,732									218,473	3,738,194	1,246,065			Kansas.
Kentucky*	3,830,636	3,745,719	2,883,295	862,424		4,354	31,116	14,740	34,707	52,198	3,372,226	406,212			Kentucky.
Louisiana	3,549,857									75,429	3,474,428				Louisiana.
Maine*	2,021,667	1,585,304	1,272,247	313,057	2,191	4,885	42,860	324,237	62,190	84,203	1,612,898		324,566		Maine.
Maryland*	2,337,334	2,073,546	1,807,892	265,654	13,411	12,366	28,236	94,946	114,829	188,284	1,719,240	[4] 429,810			Maryland.
Massachusetts*	10,859,151	9,698,037	6,454,442	3,243,595	19,181	34,074	60,555	808,740	238,564	696,272	8,078,643	925,187	1,159,049		Massachusetts.
Michigan*	14,955,637	13,982,313	10,515,682	3,466,631	155,514	11,220	105,561	181,038	519,991	251,345	7,504,292	6,000,000	1,200,000		Michigan.
Minnesota*	9,238,683	9,165,327	7,555,186	1,610,141	11,930	9,133	37,467	7,849	6,977		5,469,683		c3,769,000		Minnesota.
Mississippi*	1,600,000	1,530,785	1,372,441	158,344	25,500	476	43,239			48,000		1,552,000			Mississippi.
Missouri	6,885,456									238,755	2,611,166		4,035,535		Missouri.
Montana*	1,029,383	895,605	751,875	143,730		1,178	33,825	626	98,149	26,670		989,595		[5] 13,118	Montana.
Nebraska*	3,326,374	3,243,890	2,782,662	461,228	3,287	3,215	36,743		39,239	83,159	972,965	2,270,250			Nebraska.
Nevada	188,585										108,585		80,000		Nevada.
New Hampshire	933,157	741,788				6,343	7,030	144,664	33,332	87,542	845,615				New Hampshire.
New Jersey*	10,437,134	7,889,584	4,724,846	3,164,738	49,945	11,638	62,780	1,962,915	460,272	239,315	6,020,319	4,140,000		[6] 37,500	New Jersey.
New Mexico*	379,391	359,931	291,623	68,308	2,327	504	6,688		9,941	32,108	231,522	115,761			New Mexico.
New York*	26,082,449	24,056,668	16,820,010	7,236,658	53,570	65,568	153,427	1,585,586	167,630	1,187,386	18,671,298	6,223,765			New York.
North Carolina[1]	8,630,754									300,000	8,330,754				North Carolina.
North Dakota*	1,421,338	1,400,594	1,247,112	153,482		1,201			19,543	150,000	570,669	570,669		[7] 130,000	North Dakota.
Ohio	8,516,811	8,308,430					73,040	10,133	125,208		4,326,076	4,190,735			Ohio.
Oklahoma	4,996,420									39,215	1,959,353	2,997,852			Oklahoma.
Oregon*	5,469,761	5,349,179	4,591,136	758,043		11,283	28,000	44,737	36,562		4,102,321	1,367,440			Oregon.
Pennsylvania*	21,208,912	17,471,798	12,014,666	5,457,132	32,888	32,659	315,288	1,371,744	1,984,535	1,102,837	19,914,604			[8] 191,471	Pennsylvania.
Rhode Island*	1,680,595	1,483,744	1,090,499	393,245	662	4,031	13,990	129,080	49,088	79,057	1,601,538				Rhode Island.
South Carolina*	1,782,898	1,705,680	1,451,138	254,542	14,274	927	22,315		39,702	135,917	1,646,981				South Carolina.
South Dakota*	2,399,567	2,372,732	2,104,530	268,202		960	25,675		200	47,991	1,199,784	1,151,792			South Dakota.
Tennessee*	3,166,140	3,143,813	2,369,991	773,822		2,087	20,240			39,795	3,126,345				Tennessee.
Texas	13,046,170	12,777,253				8,755	54,221	24,786	181,155		9,429,278	3,616,892			Texas.
Utah*	576,803	552,841	439,269	113,572	2,230	1,086	12,393	3,993	4,260				576,803		Utah.
Vermont*	1,502,226	1,308,306	1,128,117	180,189	486	3,611	26,298	163,525			1,502,226				Vermont.
Virginia*	4,129,297	3,834,186	3,239,714	594,472	3,780	5,780	53,398	27,961	204,192	148,132	3,981,165				Virginia.
Washington*	5,229,759	5,092,999	3,988,395	1,104,604	34,325	12,470	85,741		4,224	182,097	4,179,142	868,520			Washington,
West Virginia*	3,254,262	2,944,126	2,473,682	470,444	2,091	4,173	49,649	98,269	155,954	240,921	1,112,756		1,900,585		West Virginia.
Wisconsin*	8,281,602	7,965,569	6,621,587	1,343,982		15,839	67,365		232,829	321,268	5,995,334	1,965,000			Wisconsin.
Wyoming*	441,035	433,364	348,500	84,864		745	6,875		51				441,035		Wyoming.
Dist. of Col.*	246,794	97,346	78,356	18,990		1,186	1,794	10,416	136,052	18,410	228,384				Dist. of Col.
Detailed total[2]	187,615,880	171,133,128	129,037,841	42,095,287	757,461	333,791	1,738,480	8,272,299	5,380,721						Detailed total.
Grand total	257,779,149									8,620,896	179,531,469	48,387,873	[9] 20,866,822	372,089	Grand total.

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[1] All States report data covering first 6 months except North Carolina, which shows full fiscal year ending June 30, 1926.

[2] Only the 36 states starred (*) in the first column reported complete receipt details which are totaled under first 9 receipt columns, shown as "detailed total."

[3] Includes $65,963 for probate judges.

[4] For Baltimore streets.

[5] For auto theft law enforcement.

[6] For toll bridge commission.

[7] Allocated to bridge fund.

[8] For State highway motor patrol.

[9] County bond payments shown above by prefix "c"

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

AUGUST 31, 1926

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1928			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,725,985.09	1298.3	$ 1,033,288.73	$ 492,916.35	75.7	$ 4,036,643.28	$ 1,865,385.81	131.4	$ 307,154.54	$ 153,577.27	19.9	$ 3,111,590.48	Alab
Arizona	10,949,878.25	5,863,772.35	729.8	272,263.37	172,004.74	26.7	1,365,684.79	944,542.22	68.8	82,189.48	50,225.99	12.2	2,586,703.70	Arizo
Arkansas	18,364,544.50	7,666,698.36	1323.0	362,027.07	179,003.72	26.7	4,364,161.43	2,065,766.25	282.0	897,052.81	443,831.94	87.9	1,250,503.74	Arkan
California	27,142,596.90	13,003,592.30	1058.0	145,035.87	87,021.52	11.5	12,159,380.20	5,922,687.78	321.3	717,759.37	416,237.89	12.3	2,643,275.51	Califo
Colorado	13,905,904.64	7,127,288.18	745.0	439,200.74	236,115.35	8.2	4,716,491.19	2,279,078.52	230.0	500,176.37	276,344.35	35.1	2,406,985.61	Color
Connecticut	5,414,567.19	2,100,585.80	117.1				4,140,337.20	1,087,698.03	59.2	583,473.04	259,391.25	4.2	886,005.92	Connecticut
Delaware	4,918,052.29	1,781,665.50	124.3				1,115,536.81	470,577.90	28.2	623,535.35	218,310.90	15.7	3,503.60	Delaw
Florida	3,832,680.26	1,824,362.32	132.9	982,922.96	489,053.29	31.9	8,702,382.31	4,034,437.58	247.9	241,489.11	119,798.25		1,617,292.56	Florid
Georgia	24,791,206.97	11,664,237.86	1794.0	710,862.89	349,830.77	39.4	12,133,040.41	5,953,775.80	625.7	827,784.58	356,571.17	20.2	107,537.40	Georgi
Idaho	11,061,198.14	5,882,112.70	724.7	295,719.53	149,898.93	7.8	2,963,163.97	1,800,754.40	177.8	792,108.22	459,819.38	71.5	267,041.59	Idaho
Illinois	44,116,611.86	20,619,995.74	1377.7	2,012,307.84	980,960.66	68.1	7,501,516.63	3,627,458.88	263.3	657,378.09	302,879.03	21.0	4,300,903.69	Illinois
Indiana	16,949,426.87	8,172,126.19	534.3	1,692,937.11	791,062.96	47.8	17,579,418.96	8,289,152.68	505.6	281,117.68	140,558.84	7.2	811,455.34	Indiana
Iowa	29,062,375.40	11,926,302.10	2114.8	378,793.37	186,807.05	25.6	11,918,971.28	5,737,713.16	733.1	2,930,081.42	1,326,666.32	119.0	308,074.38	Iowa
Kansas	32,826,601.64	12,690,489.26	1160.6	455,961.76	202,404.07	39.6	12,379,069.79	4,824,178.50	666.9	2,715,366.74	1,009,919.23	144.5	837,419.95	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	307,209.39	106,386.83	6.3	6,684,243.21	3,219,303.43	336.4	560,491.27	279,745.62	23.6	1,115,290.87	Kentucky
Louisiana	13,830,592.68	6,144,739.99	1056.9				3,497,782.62	1,697,567.74	161.3	1,346,845.99	619,114.94	47.3	810,985.33	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6				2,364,417.18	854,849.24	67.3	1,456,698.19	610,367.98	50.2	807,103.39	Maine
Maryland	10,924,943.10	6,112,991.22	423.3				513,398.28	252,643.43	30.8	1,263,113.28	559,420.96	57.7	1.49	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.5	58,723.21	22,710.05	0.2	4,807,818.26	1,284,622.27	66.1	1,174,386.11	311,928.55	19.0	1,832,204.51	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	390,946.77	188,717.04	13.6	11,635,732.67	5,332,933.17	318.1	1,034,498.96	440,238.19	23.6	2,553,424.30	Michigan
Minnesota	37,170,985.95	15,586,116.56	3181.9	1,771,684.13	821,759.07	204.0	8,004,732.22	3,003,100.00	387.2	477,555.12	36,000.00	16.1	144,804.37	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1129.0	593,316.41	294,031.56	36.8	6,865,002.79	3,401,331.84	379.0	1,321,827.50	592,714.92	49.1	425,405.58	Mississippi
Missouri	28,989,166.92	13,736,014.86	1543.2	2,937,837.46	1,263,067.36	78.8	18,107,810.63	7,230,468.54	505.5	450,972.41	202,106.41	7.5	354,778.84	Missouri
Montana	11,400,983.81	6,333,465.89	1054.9	226,140.57	134,452.39	31.9	2,360,606.34	1,856,271.18	160.7	982,999.25	580,430.98	112.2	4,520,264.56	Montana
Nebraska	11,533,401.62	5,474,202.52	1768.3	416,979.92	206,210.54	42.2	13,456,039.61	6,644,568.45	1397.9	868,508.28	410,550.09	121.2	1,899,703.40	Nebraska
Nevada	7,558,196.51	5,130,934.69	538.8	1,219,911.65	1,062,768.79	115.0	2,214,266.87	1,894,315.49	274.1	41,715.60	36,592.91		670,603.22	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6				1,339,869.53	601,785.70	39.3	38,717.88	29,092.93	0.1	161,163.30	New Hampshire
New Jersey	16,346,301.01	6,098,342.21	290.3	156,206.43	63,485.76	4.0	8,295,562.92	2,763,486.67	48.1	1,441,364.16	289,789.17	18.1	252,316.19	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1427.0				1,883,157.63	1,241,284.96	123.0	552,060.36	413,314.99	56.0	1,978,128.67	New Mexico
New York	43,224,279.79	17,911,957.19	1197.0	1,875,003.91	676,396.18	42.0	35,030,973.00	9,446,955.20	597.2	8,723,050.00	1,900,000.00	118.0	4,109,886.43	New York
North Carolina	27,009,419.47	11,177,337.94	1257.9	3,621,683.44	1,412,891.77	73.3	4,970,040.28	2,268,851.50	132.6	972,106.70	456,189.10	40.5	401,935.69	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2193.1	165,795.69	82,897.68	22.2	7,747,244.18	4,044,457.00	979.1	1,141,361.65	587,665.63	178.2	1,778.91	North Dakota
Ohio	47,699,532.90	17,371,787.03	1364.1	1,029,433.36	464,851.87	40.7	12,181,657.75	4,672,653.58	353.8	3,438,761.00	1,178,882.13	96.1	2,043,621.39	Ohio
Oklahoma	28,247,950.33	13,159,999.16	1178.9	426,315.32	212,294.12	19.6	2,613,646.38	1,215,306.60	101.8	1,285,313.86	606,754.67	96.7	865,432.46	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	357,712.64	201,299.62	22.0	3,238,079.35	1,770,139.62	113.2	238,968.69	143,381.21	13.3	171,311.76	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1188.8				27,087,825.96	7,711,872.53	550.1	2,407,226.26	731,213.01	47.6	1,334,963.42	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	207,117.54	86,175.00	5.7	1,282,981.70	340,980.00	22.7	743,891.14	201,195.00	13.4	480,389.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1481.9	698,097.01	247,788.28	38.1	6,914,916.16	2,726,699.16	204.4	155,548.99	50,921.58	19.9	9,792.06	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2181.2	285,742.04	142,871.01	78.7	3,843,913.49	1,936,835.85	639.1	596,224.95	313,144.19	99.6	170,111.98	South Dakota
Tennessee	21,624,631.57	10,276,684.02	780.0	51,931.02	27,983.60	0.9	8,873,923.50	3,952,348.28	253.1	739,242.79	328,777.17	19.0	694,897.93	Tennessee
Texas	69,183,673.48	27,440,254.72	4920.2	631,891.52	268,965.47	48.3	18,511,166.40	8,182,775.90	864.2	2,477,214.12	1,130,036.92	61.6	3,584,397.99	Texas
Utah	8,253,178.03	5,098,440.68	546.4	72,006.59	53,284.87	9.1	1,760,452.52	1,330,318.86	158.1	864,739.48	658,542.65	53.5	678,191.94	Utah
Vermont	4,242,042.64	2,017,699.51	134.6				2,098,553.60	812,423.54	39.0	51,449.37	13,905.01	0.8	424,478.94	Vermont
Virginia	21,990,249.44	10,385,728.11	1005.6	517,237.89	238,128.61	14.9	6,431,746.40	2,750,981.14	186.4	138,973.67	68,172.20	2.6	58,503.94	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	267,789.25	101,642.49	4.3	3,294,245.95	1,630,600.00	43.6	1,008,163.90	374,000.00	43.7	256,624.05	Washington
West Virginia	9,473,716.44	4,141,062.65	392.9	467,126.16	193,696.41	12.5	4,805,344.41	1,869,689.67	124.7	1,594,741.81	709,692.13	62.8	438,370.14	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1592.1	101,729.42	50,664.71	9.3	8,154,614.98	3,927,945.62	376.2	742,702.15	339,241.00	25.2	2,728,257.94	Wisconsin
Wyoming	10,928,302.66	6,040,887.05	1133.6	341,141.32	218,189.00	47.9	2,842,290.94	1,795,908.53	213.8	112,844.59	84,159.81	24.3	427,129.61	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	966,692,834.36	426,178,703.68	52,526.6	27,978,021.28	13,160,699.48	1,431.3	358,840,753.79	152,882,117.36	14,604.9	52,606,936.28	20,821,413.75	2,189.1	58,332,065.83	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 89,526,921.22 Federal aid $ 38,486,599.58 Miles 3,457.4

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 8 ▽ OCTOBER, 1926

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STEAM SHOVEL LOADING HAS BEEN FOUND TO BE ONE OF THE CHEAPEST METHODS IN QUARRY OPERATION

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1926

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U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 7, NO. 8 OCTOBER, 1926 H. S. FAIRBANK, Editor

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SIMPLIFIED SOIL TESTS FOR SUBGRADES AND THEIR PHYSICAL SIGNIFICANCE

Reported by DR. CHARLES TERZAGHI, Massachusetts Institute of Technology, Research Consultant to the Bureau of Public Roads

ONE OF the outstanding and fundamental problems of subgrade investigation consists in finding methods for identifying soils encountered in a certain locality with other soils obtained from other sections of the country. Suppose an experimental road has been built and a careful investigation has been made of how the road surface behaved under varying conditions of temperature and traffic. All these results are practically worthless unless we are in a position to determine where else in the country identical subgrade conditions exist.

The problem of identifying for engineering purposes the materials of which the soils consist is practically solved, inasmuch as we have already an experimental method which furnishes all the data required. These data concern the compressibility, the swelling, the permeability and the compressive strength of the material. They are in every respect comparable to the data used in structural engineering for expressing the properties of construction materials, and there is no doubt about their physical meaning. The method has recently been developed by the writer, and a set of equipment required for making the tests is being installed at the experimental station of the Bureau of Public Roads at Arlington, Va.

However, since the identification tests last one week for every set of six samples, it is desirable to develop methods for rapid preliminary classification of soils with a view of selecting merely some typical representatives for the more elaborate physical test.

Among the many tests which have been proposed for the preliminary classification of soils, the method devised by Atterberg seems to be by far the most promising one for the following reasons:

(*a*) The tests can be made within a short time, with simple equipment.

(*b*) They furnish three independent coefficients, viz, the liquid limit, the plastic limit, and the shrinkage limit.

However, before we can discuss the relative importance of these tests and the part they are called to play among the methods for soil identification we must find out what these tests mean and what are the factors which determine their results.

The fundamental principle of Atterberg's method is very simple and very logical. Suppose we mix a soil with a considerable quantity of water, thus transforming it into what is called the liquid state. In this state the soil is even apt to flow out like a thick juice whenever it has an opportunity to do so. However, if the water is allowed to evaporate, the consistency of the soil becomes stiffer. It first passes into a state in which it does not flow any more. However, by taking a piece of the soil and working it with the hand, we can readily mold it. We say the soil has become plastic. Further evaporation causes the soil to lose this property and to break whenever we try to mold it. Its state has become semisolid. Finally, its color changes from dark to light and it becomes very hard. (Solid state). Experience shows that the water content at which a soil passes from one of these states into the next one is very different according to the nature of the soil Hence it was plain common sense which suggested the working out of certain (although more or less arbitrary) standard tests for tracing the limit between the different states and for characterizing the soils by the water contents which correspond to the limits.

The most important disadvantage of Atterberg's, as of all the other "simplified" soil tests, as moisture equivalent, etc., is that their physical meaning is very complex, to such an extent that it can not possibly be expressed in terms of space, mass and time.

The complexity of the physical character of the tests makes it very difficult, if not impossible, to exclude from them certain arbitrary elements depending on the judgment of the observer. In addition to this, the relation which exists between the results of such tests and the mechanical properties of the soil (bearing capacity, pressure-moisture relations, permeability) is necessarily somewhat obscure and can not possibly be revealed except by experience. For these, and merely for these reasons, the preliminary investigations must inevitably be combined with a more elaborate physical one, whose outcome furnishes figures with a simple, well-defined meaning comparable to the data used in structural engineering.

THE PHYSICAL MEANING OF THE SIMPLIFIED TESTS

The purpose of this paper is to explain the physical meaning of the most important simplified soil tests and to evaluate both their possibilities and limitations.

The liquid limit.—It has been pointed out by Atterberg that the lower liquid limit is a rather arbitrary one, because there seems to be no definite reason why the test should be made precisely as Atterberg suggested. However, during the last several years experience has shown that the water content which corresponds to Atterberg's lower liquid limit has a singular significance. Suppose a clay or mud deposit has been formed by sedimentation in a pool or in a lake. At the beginning the deposit is liquid. However, as time passes the water content of the deposit gradually decreases and finally becomes constant. For this final state it has been found that the water content of the top layer of the deposit is approximately equal to the liquid limit.

Physically, the lower liquid limit merely means the water content at which the soil grains still have a certain degree of liberty to readjust themselves under the influence of slight vibrations without changing partners freely. For equal materials the lower limit increases with decreasing grain size (decreasing "specific grain number"). At equal grain size the lower liquid limit is the higher the more scale-like particles the soil contains. Hence, the value of the lower liquid limit may indicate different properties according to the character of the soil, and nothing can be told about its significance unless one knows the other data of the soil.

The plastic limit.—The plastic limit represents the lowest water content at which the soil can still be worked into threads with a diameter of one-eighth of an inch without breaking into pieces. The difference between the lower liquid and the plastic limit has been called by Atterberg the plasticity index. The greater the plasticity index the more plastic the material should be.

On hearing these statements one can not avoid a question as to what they mean. There can not be any doubt about the significance of the term "plasticity." As used in Atterberg's definitions, it merely means the capacity of the soil to undergo certain important changes in shape (changes involving a complete rearrangement of the particles) without a noticeable change in volume. Thus, in contrast to the clays, the sands are not plastic, because every appreciable change in shape of a mass of sand is associated with a very considerable increase in the volume of the sand.

Hence the controversial part of the statement does not concern the plasticity in itself, but the degree of plasticity. The striking difference between the attitude assumed by different investigators toward Atterberg's plasticity index is essentially due to two facts:

(*a*) Failure to define clearly the meaning of "degree of plasticity," and

(*b*) The hopeless attempt to correlate with each other the plasticity of different substances.

The term "degree of plasticity" has as much or as little meaning as has the term "degree of fluidity" of liquids. The degree of fluidity can be viewed from two different angles, according to whether one considers the limits of the range within which the substance is liquid (freezing point and boiling point) or considering the viscosity of the liquid within the range of fluidity (viscosity plotted against temperature). In a perfectly analogous way we can either consider the range of moisture within which a soil is plastic (Atterberg's method, upper and lower limit of plastic state) or we can consider the stiffness of the plastic material (ultimate compressive strength plotted against moisture content). Hence, if the problem is considered in such a way, there is no doubt left as to what the terms "plasticity" and "degree of plasticity" mean. The causes of plasticity do not even appear in these equations.

The attempt to correlate with each other the plasticity of different substances is, by necessity, hopeless because the plasticity of different substances may be due to essentially different physical causes. Thus the plasticity of metals is due to the formation of sliding planes across the crystalline grains; that of wax is due to the amorphous structure of the substance; of clays, to the presence of scalelike particles; and of fresh mortars, to some other still unknown causes. For this very reason the study of the plasticity of different materials requires different experimental methods. Thus, while Atterberg's method seems to be most satisfactory for soils, it fails, for instance, applied to the plasticity of mortars.

PHYSICAL SIGNIFICANCE OF THE PLASTIC LIMIT

According to what has preceded, the lower plastic limit represents the water content at which a soil ceases to be plastic, the term "plastic" meaning merely the capacity of the soil to be rolled out into threads with a certain standard diameter. However, experience has shown that the lower plastic limit has, for clay soils, a still deeper physical significance.

According to the writer's observations, the coefficient of permeability of a homogeneous clay soil decreases rapidly with decreasing water content, until, at the plastic limit, it becomes practically equal to zero regardless of the value of the plastic limit. Furthermore, the speed at which the water evaporates from the surface of a clay sample is about 4 per cent greater than the speed with which it evaporates from a free water surface, provided the moisture content is higher than the plastic limit. At the plastic limit the "relative speed of evaporation" starts to decrease and, for still lower moisture contents, this speed rapidly decreases with decreasing moisture content. From these and other facts it has been concluded that, for moisture contents equal to or smaller than the plastic limit, the physical properties of the water are no longer identical with those of free or of ordinary water.

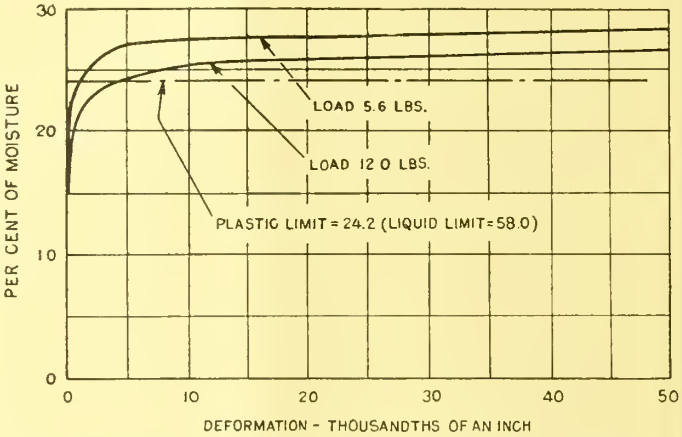

FIG. 1.—DEFORMATION DIAGRAM FOR A YELLOW BOSPHORUS CLAY SOIL SHOWING HOW THE DEFORMATION OF A CUBE OF THE SOIL PRODUCED BY A GIVEN LOAD INCREASES SHARPLY AS THE MOISTURE CONTENT PASSES THE LOWER PLASTIC LIMIT

Plasticity index (plastic and friable soils).—According to Atterberg the plasticity index is equal to the difference between the liquid and the plastic limit. The greater the plasticity index the greater is the quantity of sand with which the soil can be mixed without losing the property of being plastic. For this and for other reasons, Atterberg considered the value of the plasticity index as a standard for the degree of plasticity of the soil. According to the value of this index, he distinguished between—

Friable soils (plasticity index less than 1).
Feebly plastic soils (Index 1 to 7).
Medium plastic soils (Index 7 to 15).
Highly plastic soils (Index greater than 15).

There is no question about the validity of this classification, provided one accepts the definitions of plasticity set forth in the preceding paragraph. Due to complexity of the factors on which the phenomenon of plasticity depends, a classification based on the degree plasticity is certainly not fit for permanent purposes. However, in connection with a preliminary survey of the properties of a group of soils, it is fully satisfactory.

RELATION BETWEEN PLASTIC LIMIT AND CRITICAL BEARING POINT

Suppose we have a layer of clay, laterally confined, with a moisture content equal to the liquid limit and we put that layer under pressure so that the water squeezed out by the pressure can readily drain away. With increasing pressure the moisture content decreases. At a pressure of about 2 to 8 atmospheres (according to the character of the soil) the moisture content passes the plastic limit and the clay becomes semisolid. If at a certain pressure we interrupt the test, remove the pressure rapidly, at the same time preventing the clay from becoming resaturated, we obtain a clay-water mixture with a certain cube strength and a certain bearing capacity, both depending on the intensity of the pressure at which the test was stopped and on the character of the soil.

The relations which exist between pressure, water content, and cube strength are already so well known that we can theoretically compute what the outcome of such tests would be. Thus we may ask the following question: Suppose we reduce the water content of a soil from an initial value w_0 to a moisture content, w, less than w_0. Then we cut out of the soil a cylindrical body with a diameter of 1 inch and a height of 1 inch, and load it with a load of 12 pounds. What is the compression of this cylinder in thousandths of an inch? And what relation exists between this compression and the moisture content, w?

Let q = 12 pounds, the load acting on the cylinder,
w = the water content (in cubic centimeters per cubic, centimeter of solid material),
s = the compression of the cylinder, in inches per inch,
e = the base of the natural logarithms, and
a, c and p_0 = constants of the soil.

Based on the soil equations previously published by the writer,[1] we obtain for the compression s the formula

$$s=0.0294\,\frac{q}{e^{\frac{c-w}{a}}-p_0}+1.95\left[\frac{q}{e^{\frac{c-w}{a}}-p_0}\right]^3 \text{------} (1)$$

Figure 1 shows the relation which exists, according to this formula, between w', the moisture content in per cent of the dry weight of the sample, and s for a fat, yellow pottery clay, with a liquid limit of 58.0, a plastic limit of 24.2 and a shrinkage limit of 14.0. To substitute w' in the formula instead of w it is necessary to make use of the relation $w'=\frac{w}{n}$, in which n is the specific gravity of the material. For low moisture contents the compression of the cube is very small. However, for moisture contents above the plastic limit the compression rapidly increases with the moisture content. The sharp break in the curve corresponds approximately to the plastic limit. Since the bearing capacity of plastic soils is simply proportional to their cube strength, we would obtain for the relation between the moisture content and the penetration of a cylindrical bearing block a similar set of curves. The reason why the sharp break in the curve approximately coincides with the plastic limit is merely that, for most of the plastic soils, the pressure-moisture content curve rapidly flattens out at moisture contents below the plastic limit. It is what we may call a purely mechanical coincidence, without any deeper physical significance, but it seems to hold fairly universally.

Mr. B. H. Levenson, of the Bureau of Public Roads, has realized the same fact by pure experience. He calls the moisture content, which corresponds to the sharp bend in the curve, the "critical bearing point," because for moisture contents above that point the compressions produced by concentrated loads rapidly increase with increasing moisture content. As a result of his tests, he has found that the "critical bearing point" approximately coincides with the plastic limit for all the soils he has tested.

In addition to this, Mr. Levenson has brought out another interesting point. The formula (1) and Figure 1 refer to a clay which does not contain any appreciable quantity of air; i. e., whose interstices are completely filled with water. In contrast to this, Mr. Levenson experimented with clays which consisted of a mixture of clay, water, and air. Such clays can be considered as accumulations of moist grains which slide on each other, wherever they touch, while for the most part the voids between the grains are filled with air. It is obvious that the bearing capacity of such materials depends largely on the manner in which the samples have been prepared. Nevertheless, for such samples, too, the critical bearing point is found to coincide more or less with the plastic limit.

Mr. Levenson prepared his specimen in the following way: For compression tests with unconfined soil cylinders (a) he dried 100 grams of soil, powdered the sample, and passed it through a 2-millimeter sieve. The fraction which passed the sieve was mixed with the moisture he wanted to add; the mixture of water and clay was left over night in the humidifier, and the specimen was moulded the following morning under a pressure of 70 pounds per square inch, acting during a period of two minutes. The specimen had a height of 1 inch and a cross-section area of 1 square inch. During the compression tests the top surface of the specimen was covered with an aluminum bearing block with a diameter of 1⅛ inches and a thickness of one-eighth inch.

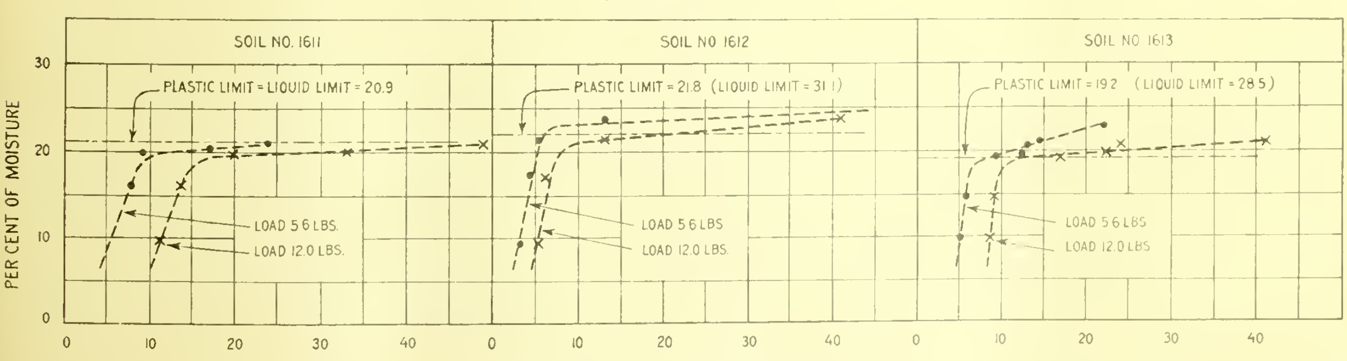

FIG. 2.—DEFORMATION DIAGRAMS FOR THREE MEDIUM PLASTIC SOILS TESTED AT ARLINGTON. THE CURVES SHOW THE DEFORMATION OF CUBES OF THE SOILS UNDER PRESSURES OF 5.6 POUNDS, RESPECTIVELY, WITH VARYING MOISTURE CONTENT. NOTE THAT THE SHARP BREAKS IN THE CURVES (CRITICAL BEARING POINTS) COINCIDE CLOSELY WITH THE LOWER PLASTIC LIMITS

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[1] Terzaghi, Erdbaumechanik, 1925.

The tests with plastic soils confined within a rigid ring (*b*) required about 300 grams of dried soil. Compacting of the sample was accomplished by a pressure of 25.1 pounds per square inch (1.76 kg. per sq. cm.), acting during a period of three minutes. The ring had a diameter of 2¼ inches and a height of 1 inch. The test was performed by placing on top of the specimen a circular bearing block covering an area of 1 square inch and applying the load on top of this block.

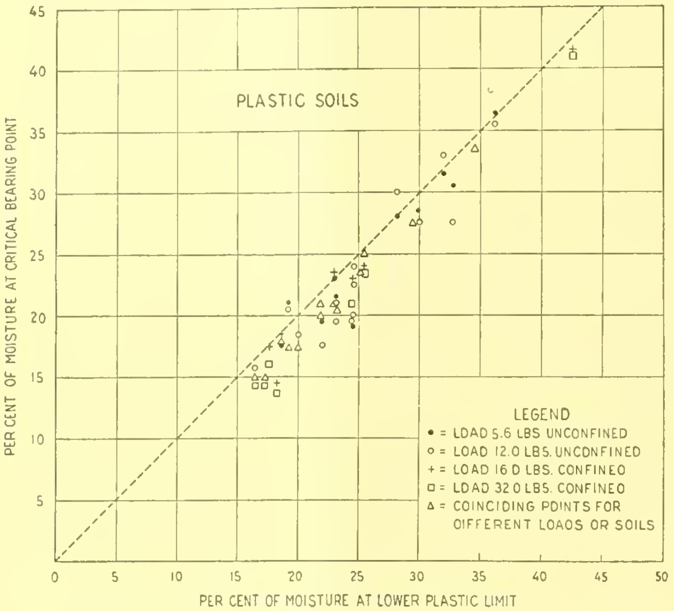

FIG. 3.—RELATION BETWEEN THE PLASTIC LIMIT AND THE CRITICAL BEARING POINT OF PLASTIC SOILS TESTED AT ARLINGTON. FOR MEDIUM AND FEEBLY PLASTIC SOILS THE CRITICAL BEARING POINT IS EQUAL TO OR SOMEWHAT LOWER THAN THE PLASTIC LIMIT

The pressure used for preparing the plastic soils for the confined tests (*b*) is somewhat smaller than the pressure required for reducing the water content of an immersed specimen to the plastic limit. Hence for moisture contents above the plastic limit, the process furnished homogeneous samples, practically identical with those referred to in Figure 1. On the other hand, for moisture contents below the plastic limit the experimenter obtained what we may call a conglomerate of wet grains, which can not be compared with those whose properties are described by Figure 1.

Some of the results obtained by Mr. Levenson are represented in Figure 2. For this figure the ordinates represent the moisture (in per cent of the weight of the dry material) with which the samples have been prepared (unconfined samples, method a). The abscissae give the compression of the soil cylinders under the influence of a load of 5.6 and 12.0 pounds per square inch, respectively, in thousandths of an inch. The upper parts of the curves run almost horizontally, as do the upper parts of the theoretical curves in Figure 1. The steep branches of the curves rise under a very much smaller angle than those in Figure 1, due to the granular structure of the soil. In spite of that the sharp break (critical bearing point) coincides closely with the plastic limit.

For very plastic soils of the type to which Figure 1 refers the critical bearing point seems to be somewhat above the plastic limit. For medium and for feebly plastic soils the critical bearing point is equal to or somewhat lower than the plastic limit (fig. 3). For friable soils the plastic and the liquid limit are identical, and their critical bearing point is approximately equal to 75 per cent of the water content which corresponds to the lower liquid limit. (See fig. 4.) Hence, if passing from the very plastic down to the very friable soils, the critical bearing point gradually shifts from the plastic range into the semisolid range.

Figure 5 illustrates the effect of a sand admixture on the bearing properties of a plastic soil. Figure 5a corresponds to a plastic soil, Figure 5b represents the same soil mixed with 50 per cent of sand (pure quartz sand, passing the 100-mesh screen), and Figure 5c corresponds to the soil mixed with 90 per cent of sand. Since an admixture of sand invariably lowers the plastic limit, the critical bearing point also goes down with increasing sand content.

THE SHRINKAGE LIMIT

If the water content of a soil decreases on account of evaporation to values below the lower plastic limit, the soil is said to pass from the plastic into the semisolid state. During this process the decrease in volume of the soil is precisely equal to the value of the water lost by evaporation. However, as soon as the moisture content reaches a certain minimum limiting value the volume of the sample ceases to diminish, while the weight of the sample still continues to decrease. We say that the sample has passed from the semisolid into the solid state. The limit between the two states is marked by a change of the color of the sample from dark to light. The moisture content which corresponds to the point of transition between the two states was called by Atterberg the "shrinkage limit."

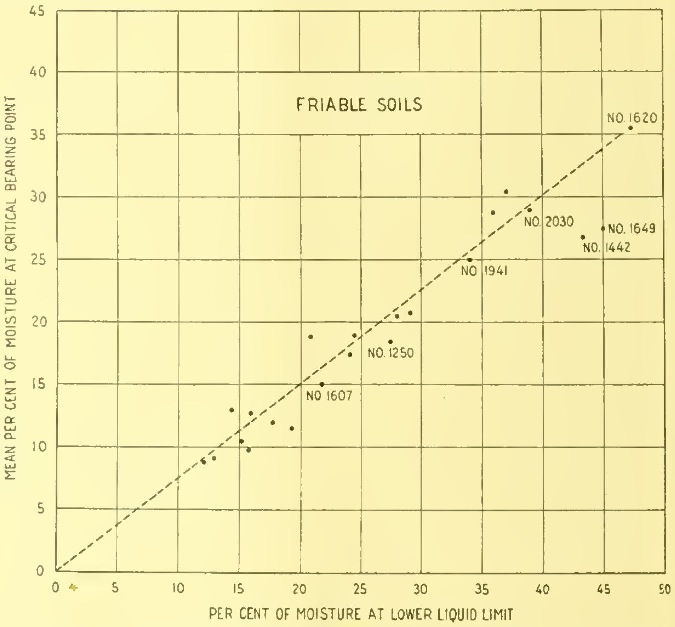

FIG. 4.—RELATION BETWEEN THE LOWER LIQUID LIMIT AND CRITICAL BEARING POINT OF FRIABLE SOILS TESTED AT ARLINGTON. FOR FRIABLE SOILS THE CRITICAL BEARING POINT IS APPROXIMATELY EQUAL TO 75 PER CENT OF THE WATER CONTENT CORRESPONDING TO THE LOWER LIQUID LIMIT

The mechanics of the process of shrinkage has been explained by the writer in his theory of the capillary pressure[2] in the following way:

Figure 6A shows a cross section through a small lump of soil whose moisture content is greater than the shrinkage limit. The voids are completely filled with water. The free surface of the water is located within

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[2] Terzaghi, Erdbaumechanik. 1925.

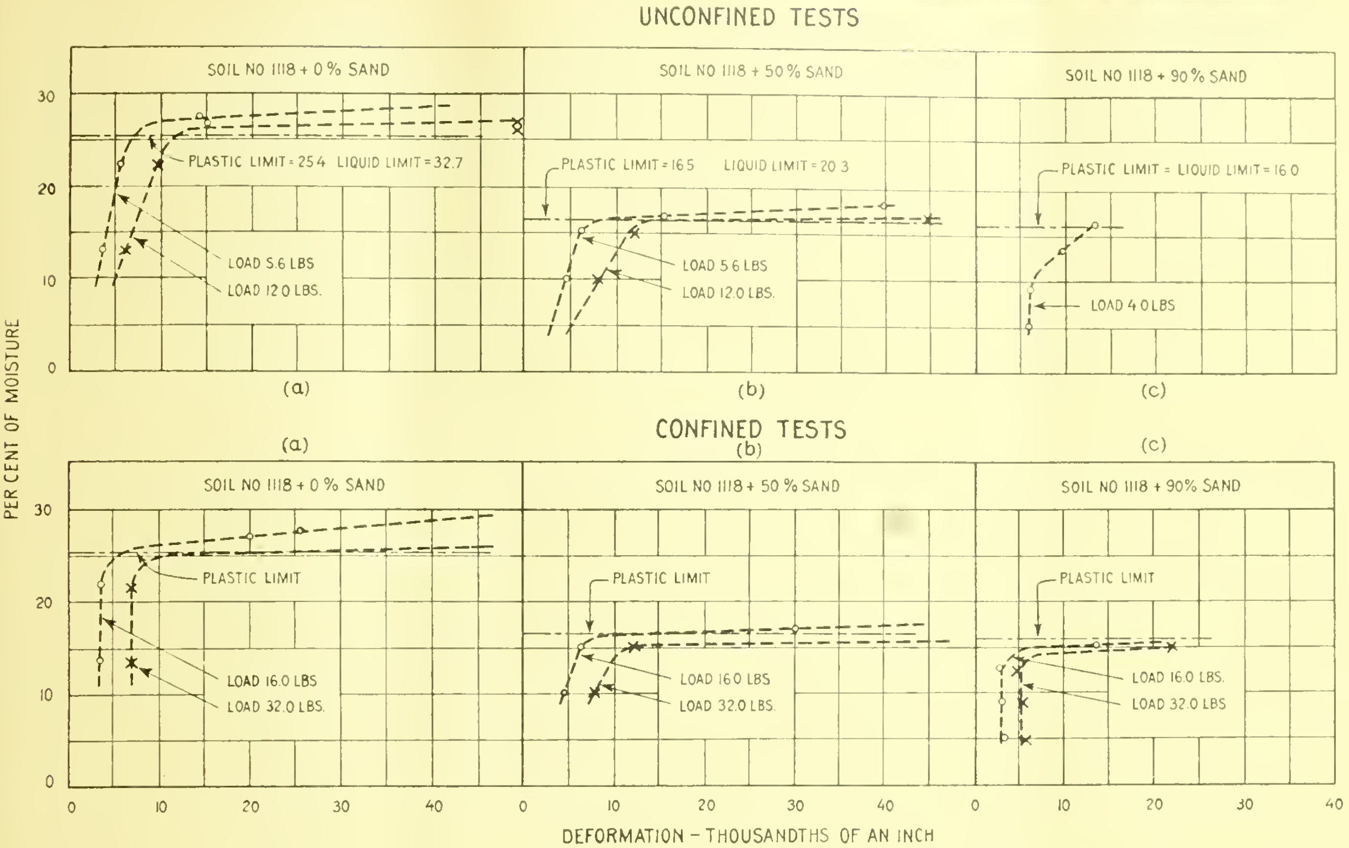


Fig. 5.—Relation of Moisture Content and Deformation of a Soil Tested at Arlington, Showing the Effect of Admixtures of Sand on the Bearing Properties of the Soil

the surface of the sample and the surface tension exerts on the outer surface of the sample a uniformly distributed pressure, in every respect comparable to an external pressure acting like a hydrostatic pressure at every point perpendicular to the outer surface of the sample. This surface tension, which causes the soil to shrink, is the same force which drives the water up in a capillary tube and which prevents the water from flowing back through such a tube, provided the length of the tube is smaller than the height of capillary rise. No air can get from outside into such a tube and, for the same reason, no air can get into the clay as long as the pressure exerted by the surface tension is smaller than the pressure required to still further compress the clay.

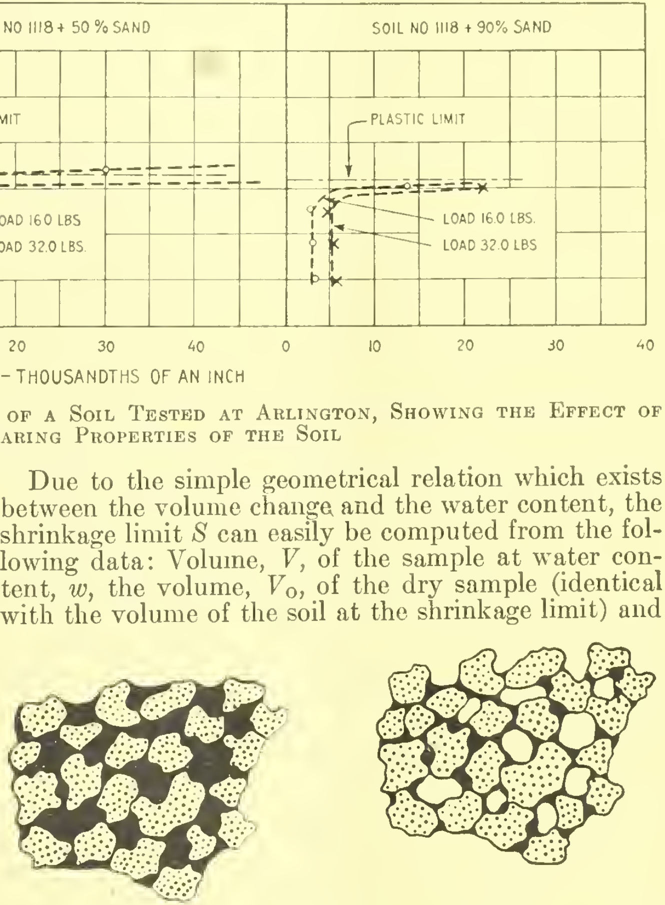
At the lower liquid limit the pressure exerted by the surface tension of the water or by the "capillary pressure," as it should be called, is practically equal to zero. At the lower plastic limit it amounts already to several atmospheres and, while the sample passes through the semisolid state, the capillary pressure becomes still higher. Finally, however, there comes a point where the force required to produce a volume change equal to the volume of the evaporating water becomes greater than the maximum value the capillary pressure can possibly assume. Hence, if still more water evaporates, the surface of the capillary water withdraws into the interior of the soil. Air enters the voids and, as a consequence, the color of the soil changes from dark to light. The water content at which this change occurs is the "shrinkage limit." The shrinkage limit obviously depends on two factors: The compressibility of the soil and the maximum value of the capillary pressure.

Due to the simple geometrical relation which exists between the volume change and the water content, the shrinkage limit S can easily be computed from the following data: Volume, V, of the sample at water content, w, the volume, V_O, of the dry sample (identical with the volume of the soil at the shrinkage limit) and the weight, W, of the dry sample. At water contents above the shrinkage limit no air gets into the sample. Hence the difference, $w-S$, between the original water content, w, and the shrinkage limit, S, is obviously equal to the volume change, $V-V_O$, divided by the dry weight, W, or

$$\frac{V-V_O}{W}=w-S$$

and therefore

$$S=w-\frac{V-V_O}{W}$$


A
VOIDS COMPLETELY FILLED WITH WATER

B
VOIDS FILLED WITH WATER AND AIR

Fig. 6.—Cross Sections of a Lump of Soil, (A) When the Moisture Exceeds the Shrinkage Limit, and (B) When the Moisture Content Is Less Than the Shrinkage Limit

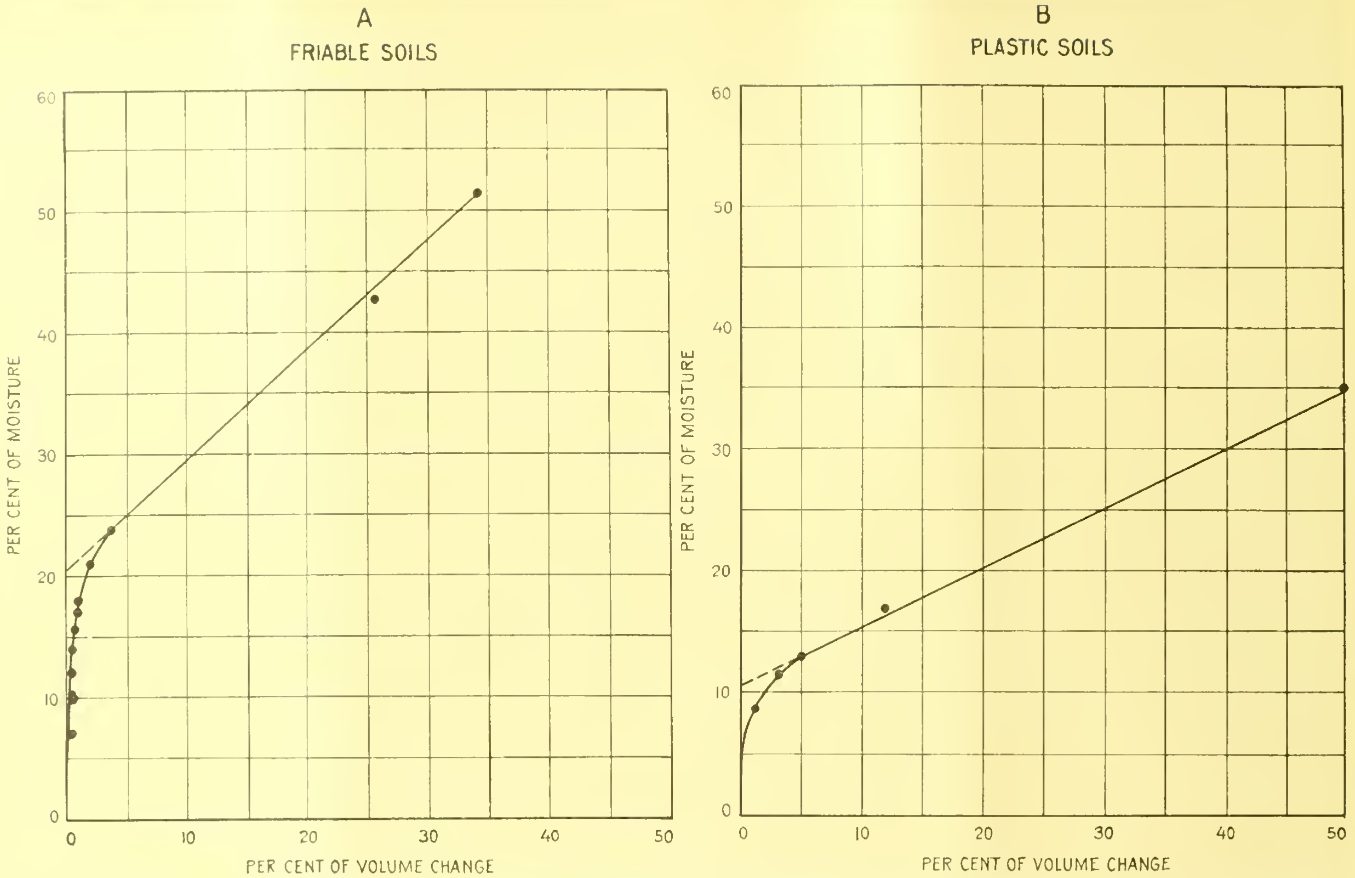


Fig. 7.—Typical Relation Between Moisture Content and Volume Change for Friable and Plastic Soils. The Shrinkage Limit Is Indicated by the Intersection of the Extended Portion of the Straight Part of the Curve With the Vertical Axis

If this equation be plotted as a diagram, one obtains for the relation between volume change and moisture content a straight line. The ordinate of the point of intersection between this line and the vertical axis is equal to the shrinkage limit (fig. 7). In practice, however, the semisolid state passes not suddenly but gradually into the solid state for the following reason: As soon as the surface of the water withdraws from the outer surface of the soil into the interior (fig. 6B), the surface tension acts no longer like an external pressure, because the capillary water forms now innumerable droplets scattered throughout the interior of the soils, separated from each other by small air bubbles. The surface tension is confined to the vicinity of the points of contact between the individual grains, and its effect essentially consists in slightly rearranging the individual particles by pulling them still closer together. The additional compression produced by this process is very small. Nevertheless, it shows up quite clearly on every moisture content—volume change diagram. (See fig. 7.)

RELATIONS BETWEEN THE LIMITS OF CONSISTENCY

From the preceding discussion of Atterberg's limits it became evident that each one of the limits depends on different factors cooperating in a different way in producing the consistency to which the limits refer. Thus the lower liquid limit essentially depends on the number of grains per unit of volume and, to a smaller extent, on the shape of the particles. On the other hand, the plastic limit and the index of plasticity largely depend on the shape of the grains while the number of grains (specific grain number) seems to play a less important part. The shrinkage limit varies with both factors and, in addition, to a large extent with the uniformity of the soil. Hence it is obvious that both the values of the limits and the relation between the different limits are apt to vary with the soil (fig. 8). In general, for friable soils, the shrinkage limit may be anywhere between the lower liquid limit and 50 per cent of its value. For plastic soils the shrinkage limit ranges between a water content of 10 and 20 per cent (in exceptional cases more), and no definite relation exists between the plastic and the shrinkage limit.

If a plastic soil is mixed with sand, both the lower liquid and the lower plastic limit decrease with increasing sand content. At the same time the difference between the two limits (Atterberg's plasticity index) decreases and, at a certain percentage, *p*, of sand, the plasticity index becomes equal to zero. According to Atterberg, *p* should be the higher, the greater the plasticity index.

THE MEANING OF THE MOISTURE EQUIVALENT

The limits of consistency described in the preceding paragraphs have always the same physical significance regardless of the fineness of the soil. In contrast to this an analysis of the moisture equivalent test has shown that the meaning of this test essentially depends on the effective size of the soil grains.

This analysis was based on the known relation which exists between the centrifugal force, the effective size of the voids, and the maximum value of the surface tension. It led to the following conclusions:

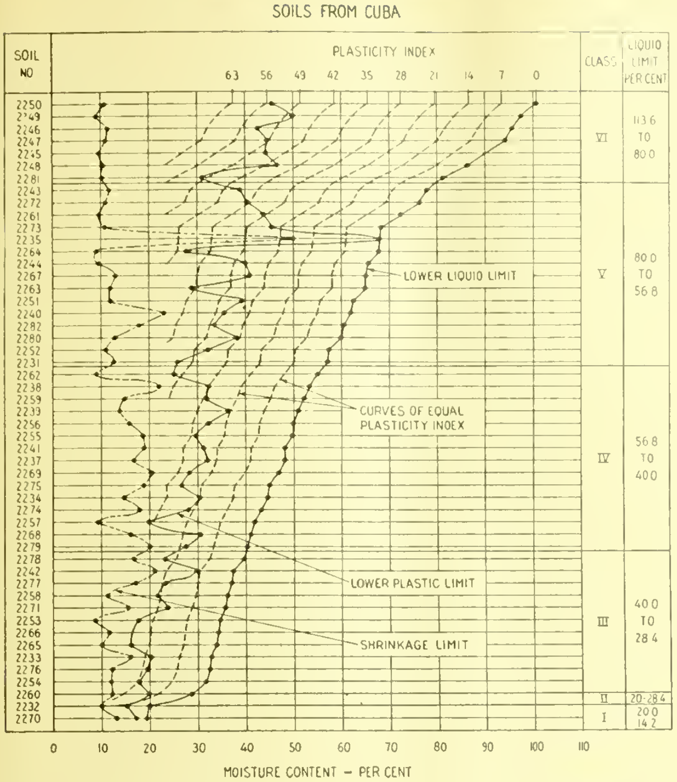


Fig. 8.—Lower Liquid and Plastic Limits and Shrinkage Limits of Soils from Cuba Tested at Arlington, Showing by the Length of Abscissa Between the Plastic and Liquid Limits the Plasticity Index of the Soils

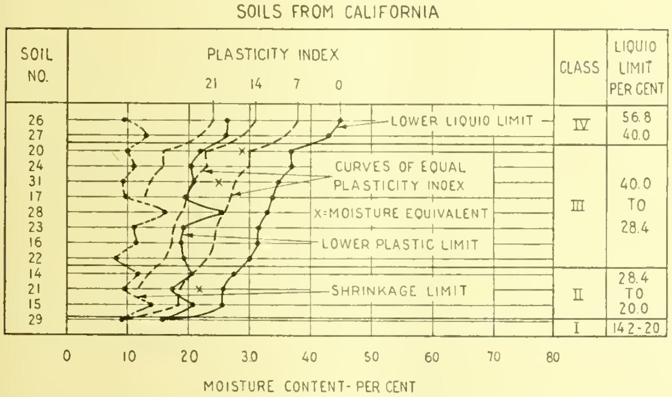


Fig. 9.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from California

(*a*) If plastic soils (ordinary clay soils) are submitted to the moisture equivalent test, the voids of the soil remain completely filled with water. The centrifugal force combined with the surface tension of the water exerts on the sample a force whose intensity depends on the speed of rotation, on the thickness of the layer of soil, and, for very plastic soils, on the duration of the test. The greater the speed of rotation the thicker the layer, and (for very plastic soils) the greater the duration of the test the smaller is the moisture equivalent. If performed under the standard condition described in Public Roads, April, 1925,[3] the moisture equivalent represents the water content which corresponds to an external pressure of approximately 2 kilograms per square centimeter, acting for a period of one hour. Since the pressure required for reducing the water content of a clay down to the lower plastic limit amounts to 2.5 kilograms per square centimeter or more, the moisture equivalent for plastic soils should always be somewhat higher than the lower plastic limit. Figures 9 to 12, inclusive, show the limits and the moisture equivalent for a great number of soils tested in the experimental station of the Bureau of Public Roads at Arlington. For all the plastic soils (i. e., for all those soils whose lower liquid limit is appreciably higher than the lower plastic limit represented on the diagrams) the moisture equivalent is located between the two limits.

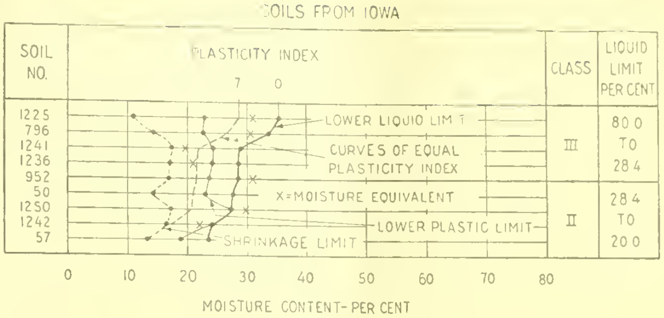


Fig. 10.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from Iowa

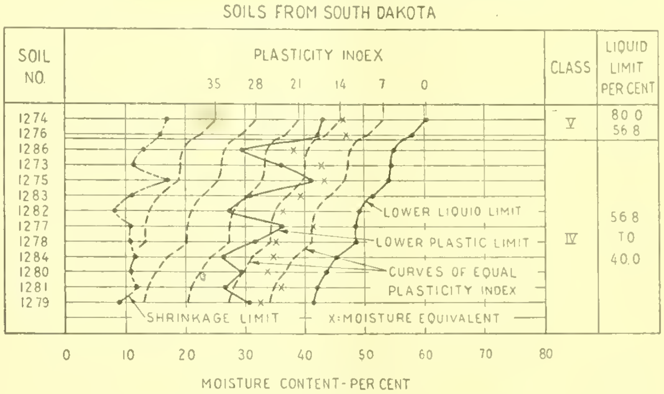


Fig. 11.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from South Dakota

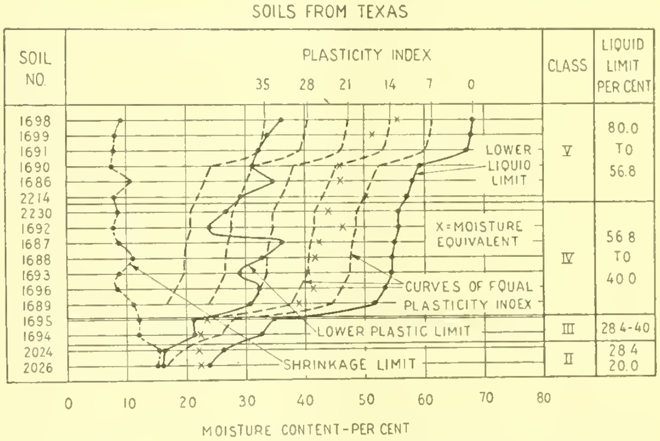


Fig. 12.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from Texas

Hence from the results of our analysis it became evident that, for plastic soils, the moisture equivalent has nothing whatsoever to do with the drainage properties of the soil. It merely tells about the compression produced by a certain pressure.

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[3] Procedure for Testing Subgrade Soils, by J. R. Boyd, Public Roads, vol. 6, No. 2, April, 1925.

(*b*) If friable soils with fairly large voids are submitted to the moisture equivalent test, the centrifugal force acting on the capillary water overcomes the surface tension. Part of the capillary water is ejected from the voids; air invades the interior of the sample and the capillary water is split up into a great number of small droplets, adhering to the points of contact between the individual grains. As a consequence, the pressure acting on the sample during the test has no more a definite value. The pressure depends on the ultimate water content which, in turn, depends on the number of points of contact per unit of volume. The greater this number the greater the moisture equivalent. Hence in this case, the moisture equivalent becomes an index of the draining properties of the soil. Since not all of the voids are filled with water, the moisture equivalent should, for friable soils, be lower than the lower liquid limit. This statement, too, is confirmed by the data represented by Figures 9 to 12, inclusive.

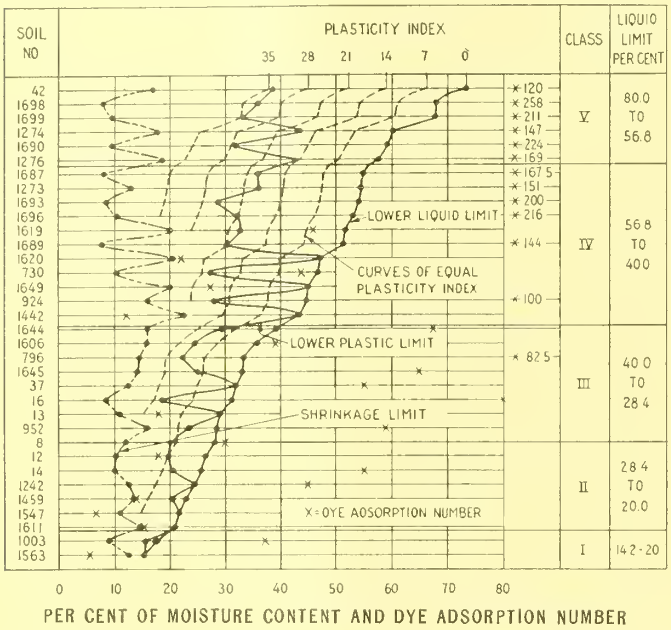

FIG. 13.—REGIONAL DIAGRAM SHOWING LIMITS AND DYE ADSORPTION NUMBERS OF SOILS

(*c*) For soils intermediate between plastic and friable soils the moisture equivalent has no definite meaning at all.

FIELD MOISTURE EQUIVALENT TEST ERRONEOUSLY NAMED

In connection with the earlier attempts to investigate the properties of subgrades, A. C. Rose, of the Bureau of Public Roads, developed a field test which was believed to furnish the moisture equivalent without using the centrifuge. He described his test as follows:[4] "The test is made by taking a 500-gram sample of air-dried soil, breaking up the lumps, placing the sample in a bowl, adding water slowly from a burette, mixing the water and soil until it reaches the consistency of putty and may be compacted with a spoon or spatula without any free water remaining on the surface. Water is then allowed to drop upon the smoothed surface as long as it is abosrbed. Before the moisture equivalent percentage is reached, the sample will absorb water readily, but when the critical value is passed, the surface will retain a wet, shining appearance."

From the mere description of the test one learns that the result has physically nothing in common with the result of the standard moisture equivalent test. For plastic soils the moisture equivalent represents the moisture which corresponds to a pressure of approximately 2 kilograms per square centimeter, provided the initial moisture content of the sample was considerably greater. In contrast to this, Mr. Rose's field-moisture determination represents the water content which corresponds to a pressure of zero, provided the initial water content of the sample was considerably lower.

In order to realize the physical meaning of the field-moisture test, one has to remember the fact that under normal conditions the capillary water contained in the voids of a plastic soil with a plastic consistency is in a state of tension. Evaporation of capillary water increases the tension, while the moistening of the sample causes the tension to drop. (See the paragraph on the shrinkage limit.) Tension in the capillary water means a negative hydrostatic pressure. On the surface of the sample, the hydrostatic pressure is equal to zero. Hence, if one places a drop of water on the surface of a piece of soil whose capillary water is in a state of tension, the water flows at once toward the zone of negative pressure, and one says the water "has been absorbed". However, by steadily increasing the moisture content one steadily decreases the tension in the capillary water until finally a point is reached where this tension becomes equal to zero. This point obviously represents Mr. Rose's field moisture, because the criterion of the test consists in the sample losing its property of absorbing additional moisture.

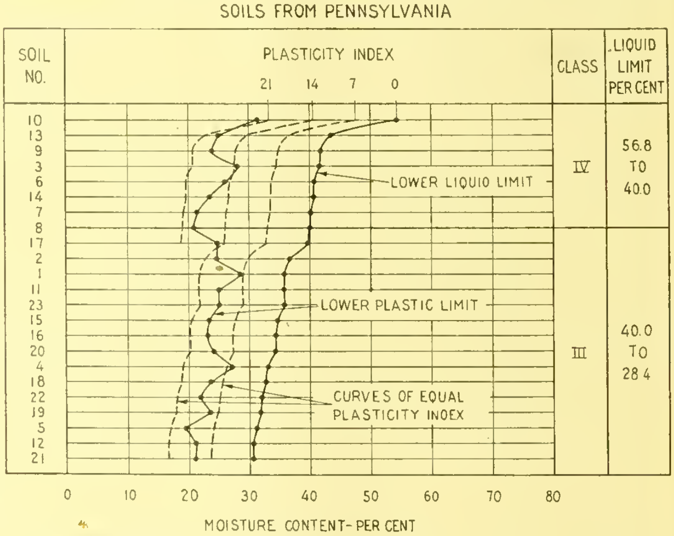

FIG. 14.—REGIONAL DIAGRAM SHOWING LIMITS OF SOILS FROM PENNSYLVANIA

Figure 15 shows the relation between pressure and voids ratio for a feebly plastic Pennsylvania soil. The initial water content of the sample was equal to the lower liquid limit. By gradually raising the pressure from zero to 3 kilograms per square centimeter (AC), the voids ratio dropped from 0.837 to 0.595. By reducing the pressure again from 3 kilograms per square centimeter to zero, the voids ratio increased again from 0.595 to 0.625 (rebound curve CD).

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[4] Practical Tests for Subgrade Soils, by A. C. Rose, PUBLIC ROADS, August, 1924.

In the diagram, the section, AB, of the upper curve represents the changes the sample would have undergone during a moisture equivalent test (raising the pressure from zero to 2 kilograms per square centimeter), and the ordinate of the point, B, represents the moisture equivalent in terms of voids ratio. In contrast to this, the field-moisture test is represented, not by a part of the upper but of the lower curve CD, and the ordinate of the point D represents the field moisture. In this particular case the "field-moisture" test would have furnished practically the same result as the moisture equivalent test (0.625 against 0.622 in terms of voids ratio). Nevertheless, the close agreement between the values would have been no less a mere mechanical coincidence than is the coincidence between the plastic limit and the critical bearing point.

Thus the "field-moisture" determination merely indicates the maximum amount of moisture a soil is apt to absorb when its moisture content is increased from a lower value by gradually adding moisture. The objectionable feature of the test consists in the fact that the "field moisture" obviously depends on the initial water content. Thus, if the test is started at the point B, of the diagram, Figure 15, instead of at point C, the sample would have passed through the process BD¹, and the result of the test would have been a voids ratio of 0.661 instead of 0.625. For this reason one can hardly expect consistent results in case the tests are made by two independent observers.

According to Mr. Rose, the soils of the Pacific Northwest were found to represent good, mediocre, or poor subgrades, according to whether their "field moisture equivalent" was smaller than 20, from 20 to 30, or greater than 30 per cent. Moisture contents of 20 and 30 per cent, respectively, correspond to the volumes of voids of 35 and 44 per cent, respectively. Considering this fact and the physical meaning of the field moisture test, Mr. Rose's statement has the following significance: A subgrade is good or mediocre according to whether a thorough soaking after a period of dryness does not increase its volume of voids beyond 35 or 44 per cent, respectively. This experience is closely related to the fact that the same sand may have a very great bearing capacity or almost none according to whether its volume of voids is equal to 42 or 48 per cent[5] (more or less, depending upon the nature of the sand). This, in turn, is connected with the fact that the volume of voids in an accumulation of equal spheres ranges between the extreme limits, 25.8 per cent and 47.6 per cent. The greater the volume of voids, the smaller the resistance of the spheres against mutual displacement, and beyond a value of 47.6 per cent the accumulation is unstable, which means that its bearing capacity drops to a very small value.

The "ultimate swell," represented by Mr. Rose's field moisture, is among the data furnished by the standard swelling test which has recently been incorporated in the investigation program of the Bureau of Public Roads. Figure 15 represents the results of such a test. Since both the initial state of the sample and the pressure corresponding to the point, C, are standardized, the personal element is excluded and the "ultimate swell" thus obtained represents a physical fact with known meaning. It will be among the problems of the proposed field survey to determine whether the ultimate swell is the only factor which governs the behavior of the subgrade or whether there are other additional factors entering into the relation between subgrade and road surface. For the time being it would be premature to express any opinion concerning the possible outcome of this investigation. However, Mr. Rose's observations represent a very valuable group of facts to start with.

THE DYE ADSORPTION TEST

The results of all those tests which have been described in the preceding paragraphs essentially depend on the mechanical properties of the soil; i. e., on the size and shape of the grains and on the uniformity of the mixture. In contrast to this, the result of the dye adsorption test seems to depend essentially on the chemical nature of the soil particles and on their electrolyte content. As a consequence, there seems to be no definite relationship between the limits and the dye adsorption number, except for the fact that the soils with high liquid limits have higher dye adsorption numbers than soils with a low liquid limit. To illustrate this statement, Figure 13 may serve. In this figure, the erratic character of the dye adsorption number is quite conspicuous, soils with similar limits having widely different dye adsorption numbers. Hence it is obvious that the dye adsorption number expresses among others a factor independent of those which determine the limits. This, in itself, is a very valuable property of the dye adsorption test. However, before we are able to make any statement concerning the value

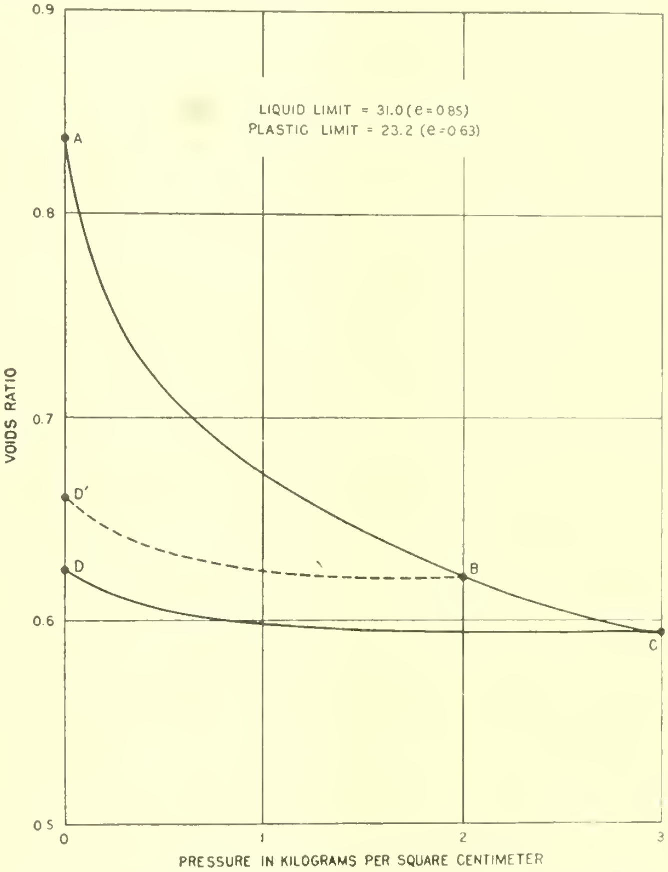

FIG. 15.—PRESSURE-VOIDS RATIO DIAGRAM FOR A FEEBLY PLASTIC PENNSYLVANIA SOIL, ILLUSTRATING THE DIFFERENCE BETWEEN THE TRUE MOISTURE EQUIVALENT (B) AND THE SO-CALLED "FIELD MOISTURE EQUIVALENT" (D, D')

of the dye adsorption test, we must know whether the factor expressed by the dye adsorption number has an appreciable effect on the mechanical properties of the soil. This question can not possibly be decided except by a systematic physical investigation of soils with similar limits, but with widely different dye adsorption numbers.

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[5] Terzaghi, Erdbaumechanik, 1925.

INTERPRETATION OF TEST RESULTS

If dealing with the interpretation of test results obtained in the laboratory, we must first of all consider two important facts. The first one consists in the soil tests being confined to that fraction of the soil which passes a certain standard mesh screen (width of opening 0.5 mm.). Hence a correction must be made for the influence of the coarser constituents on the properties of the soil. For the time being, no reliable basis for making such a correction exists and systematic investigations will be required for establishing such a basis.

The second fact consists in the structure of the undisturbed soil being different from the structure of the soil subjected to the tests in the laboratory. It is conceivable that the same soil may exhibit somewhat different properties according to whether the structure of the top layer has developed under the influence of a very cold or a very warm, a humid or an arid climate. It is one of the problems of the field survey to investigate the importance of these influences.

These two facts impose upon the interpretation of the test results obtained in the laboratory certain unavoidable limitations. The man working in the laboratory can not possibly do more than furnish the data required for identifying the raw materials of which the subgrades consist. Hence all his efforts ought to be governed by this and merely by this one intention.

Since the sole function of the subgrade consists in supporting the road surface, we are merely interested in its mechanical properties; that means in its compressive strength, its elastic properties (swelling), its permeability and in the influence of the moisture content on its resistance. The physical meaning of these properties is as well defined as is the meaning of the data used in structural engineering.

In contrast to this, the simplified soil tests have apparently no bearing on the strength of the materials except for the fact that the plastic limit approximately coincides with the "critical bearing point." Hence we are entitled to ask wherein their value resides. The answer to this question can easily be deduced from the preceding analysis of the meaning of the simplified soil tests. According to this analysis, the result of the simplified soil tests depends precisely on the same physical factors which determine the resistance and the permeability of the soils (shape of particles, effective size, uniformity) only in a far more complex manner. Thus for instance a high lower liquid limit associated with a low shrinkage limit may mean either an excessively fine-grained soil with a low swelling capacity or a more coarse-grained soil with a very marked swelling capacity. According to whether these two limits have the first or the second meaning, the plastic limit would be higher or lower. In spite of these complications, we are safe in making the following statement: If several soils with similar geological origin have fairly identical limits, their physical properties too will be fairly identical and it will be sufficient to investigate a single one of them more thoroughly. Hence the limits represent an excellent means for preliminary soil classification.

If we get the mere description of the appearance of a soil, we know about that soil practically nothing, and an identification is simply impossible. If we know the three limits of the soil, we are already in a position to compare this soil with others and can at least anticipate what its properties may be. If we know in addition the results of physical tests performed on another soil with fairly identical limits, we can say the soil is known.

REGIONAL SOIL DIAGRAMS FACILITATE CLASSIFICATION

The tabulated results of the determination of the limits for a group of soils consist of a confusingly great number of apparently incoherent figures, and it is very difficult to perceive the relationship which exists between the soils represented in the table. In order to facilitate this work, and to visualize the characteristic features of the soils, it is proposed to represent the results of the investigation in a graphical form. For this purpose it is advisable, first of all, to arrange the soils according to the value of the lower liquid limit, because this limit is by far the most variable one, ranging anywhere between 15 and more than 100 per cent.

The graphical representation is performed in the following manner: The individual soils are represented by horizontal lines, one quarter of an inch apart. On every line the limits of the soil are plotted to a scale of 1 inch equal to 10 per cent moisture content. Then all the points representing the same limit are connected with each other by continuous curves. Since the soils have previously been arranged according to the value of their lower liquid limit, the lower liquid limit curve slopes in one direction only, while the others have a more or less sinusoidal shape. The result of this operation will be called the regional soil diagram, representing the variation of the limits of soils coming from a definite region.

By tracing curves parallel to the lower liquid limit curve at horizontal distances representing 7, 15, 22 per cent, etc., from this curve, one divides the space between the liquid limit and the plastic limit curve into sections of uniform width. Since the total distance between the liquid and the plastic limit represents the plasticity index, the subdivision of this distance by parallel curves facilitates the evaluation of the degree of plasticity (according to Atterberg's definition) of the soils.

Figures 8 to 12 inclusive and Figure 14 represent the regional soil diagrams for soils from Cuba, California, Iowa, South Dakota, Texas, and Pennsylvania investigated in the laboratory of the Bureau of Public Roads at Arlington. By merely glancing at these diagrams one sees at once the difference which exists between the soils coming from the different regions. For the time being, we do not precisely know wherein the significance of the limits—for instance, the significance of a low plastic and a high liquid limit—resides, but the mere fact that there is a great difference between the average limits of the soils coming from different sections of the country plainly shows that there must also be a very considerable difference between their physical characteristics. The knowledge of the significance of the limits will automatically come as soon as a sufficient number of samples has been submitted to the standard physical tests.

(Continued on p. 170)

DIRECT PRODUCTION COSTS OF BROKEN STONE

By DR. GEORGE E. LADD, Economic Geologist, Bureau of Public Roads

THERE are no available sources of information as to the direct costs of producing broken stone which are satisfactory for comparative purposes by quarry operators. Something can be found in cost data manuals on certain operations but the material is scattered and unsatisfactory because the conditions under which the operations were conducted are usually not stated.

Some time ago the Bureau of Public Roads undertook a study of quarries and broken stone production, especially direct costs and conditions governing them. It was decided not to enter the field of overhead costs which vary largely at different quarries and in different localities, and the following items also were omitted from the study: Capital investment, interest, royalties, general repairs, insurance, depreciation, depletion, opening of quarry face, general supervision, accounts, sales, and delivery. This reduced the investigation to a study of the direct production costs which included only such supervision as that employed directly in quarry or plant.

Different types of quarries, pit and open-face, high and low-face, and large and small plants operating in various kinds of rock were selected for study and all conditions were noted that might affect costs. As it was desired to make the results of practical value at any time, quarry methods of operation, wage rates, costs of materials and labor hours, as well as dollar costs per unit were recorded. Since the results are given not only in terms of dollars but also in terms of labor hours and materials they should be of permanent value to quarry operators and all others engaged in rock excavation.

METHOD OF STUDY

The operation of producing broken stone was divided into 10 natural units as follows: Stripping, drilling face, drilling bowlders, blasting face, blasting bowlders, sledging and steel balling, pumping, loading and hauling waste; loading and delivery to crusher; and crushing, screening, etc.

The data were acquired by daily visits to selected quarries for a period usually of eight weeks. Pay rolls and books, where kept, were consulted and all results were checked by personal observation. Methods of operation were studied and the rock being quarried was sampled for laboratory test to determine its nature.

A complete report on these studies, giving a description of each of the 23 quarries studied with data collected at each and comparisons of operations at the various quarries, is to be published but it is considered worth while to present at this time some of the comparisons and conclusions. The various items will be discussed in the order listed above. In studying the data presented in the different tables it may be desirable to know the scale of operation of the different quarries which are referred to by key numbers, and this information is given in Table 7.

DISCUSSION OF OPERATING COSTS

Stripping.—Stripping was conducted at 14 of the 23 quarries. At the others there was either no stripping or so little that it was shot down with the face and eliminated in the screening operation. At the 14 quarries where stripping was done the cost varied from $0.0022 to $0.0873 per ton of broken stone. With one exception all stripping was removed by hand labor and dump carts.

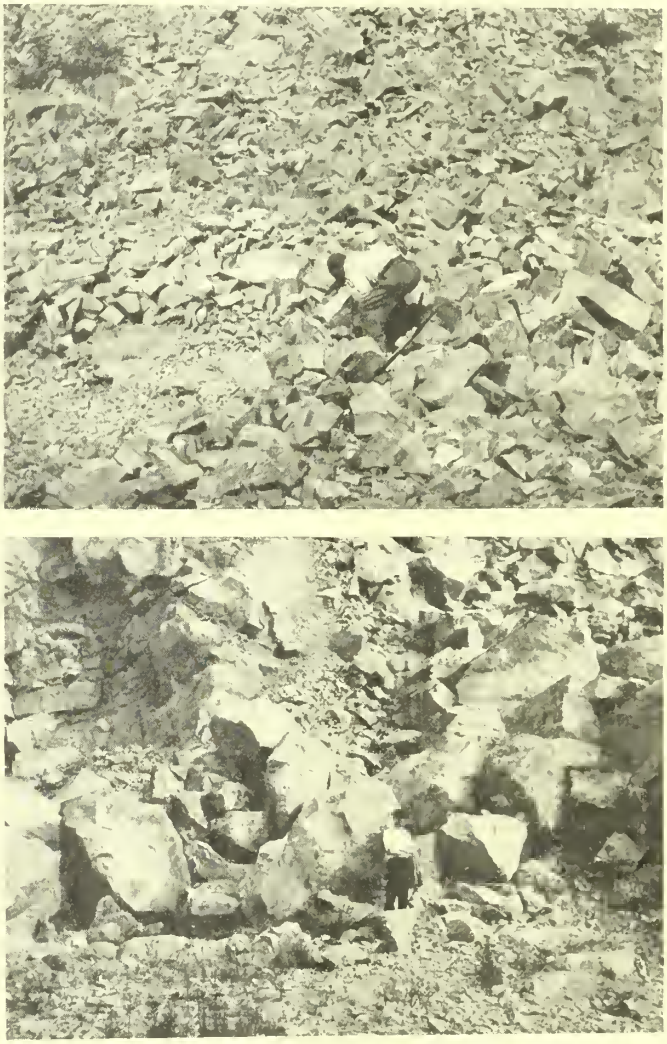


Fig. 1.—The Upper Picture Illustrates the Advantage of a Ledge (Trap) That Blasts into Relatively Small Bowlders, While the Lower One Shows a Granite Ledge That Breaks into Large Bowlders. Subsequent Bowlder Breaking Cost 1½ Cents at the First Quarry and 12 Cents at the Second

Drilling face.—Table 1 summarizes the data on face drilling. It should be noted that low cost per foot of drilling does not necessarily mean low cost of drilling face per ton of broken stone on account of wide differences in the spacing and depth of holes, and the combination of snake holing with vertical holes.

The average rate of drilling per hour was 2.4 feet for tripod drills and 1.46 feet for well drills. Comparison of rates of drilling with the laboratory results of the wear test indicates a relation between them which

TABLE 1.—*Drilling face data* [1]

Quarry key No.	Kind of rock	Kind of drill	Direct cost of drilling face per ton of broken stone	Cost per foot drilled	Average feet drilled per hour per drill	Tons down per foot drilled	Factors affecting results
18	Diabase (trap)	Tripod	$0.0247	$0.80	1.81	69.09	High face, numerous joint planes, tripods, snake holes, electricity, air.
8	Dolomite	Well	.0250	.68	1.29	27.24	High face, well drills, deep holes, far apart, electric power.
[2] 17	Diabase (trap)	do	.0264	2.06	.65	53.47	High face, well drills, hard rock, deep holes, far apart, steam power.
21	Limestone	do	.0273	.53	3.10	19.23	Medium high face, well drills, soft rock, holes far apart, electric power.
20	Dolomitic limestone	do	.0308	.74	2.48	24.53	High face, well drills, soft rock, holes far apart, gasolene power.
7	Siliceous dolomite	do	.0375	.94	.80	25.15	Do.
22	Marble	Tripod	.0470	.40	(3)	8.51	High face, tripods, soft rock, snake holes, electric power, air.
19	Diabase (trap)	do	.0487	1.57	1.61	32.25	High face, tripods, hard rock, snake holes, electric power, air.
23	Slate	Well	.0596	1.43	.91	24.09	High face, well drills, hard rock, snake holes, steam power.
3	Granite	do	.0819	1.83	1.68	18.70	High face, well drills and tripods, hard rock, oil and electricity.
10	Diorite (trap)	Tripod and well.	.1167	.76	1.20	6.48	High face, well drills and tripod (mostly), hard rock, electricity, air.
12	Altered rhyolite (trap)	Tripod	.1323	.69	2.34	5.19	Low face, benches, medium hard rock, hole spacing close, electricity, air.
15	Trachytic rhyolite (trap)	do	.1835	.42	4.13	2.31	Low face, benches, tripod, medium hard rock, hole spacing close, steam.
6	Conglomerate	do	.1845	.45	4.39	2.45	Low face, benches, tripod, medium hard rock, hole spacing close, electricity, air.
14	do	do	.2382	.95	2.54	4.00	Low face, benches, tripod, medium hard rock, hole spacing close, steam.
1	Granite	do	.2384	.87	2.05	3.68	Low face, benches, tripod, very hard rock, hole spacing close, electricity, air.
5	Rhyolite conglomerate	do	.2509	.64	3.73	2.53	Low face, benches, medium hard rock, hole spacing close, steam.
11	do	do	.2667	.62	2.49	2.33	Do.
4	Granite	do	.2685	1.57	1.96	5.82	Low face, benches, tripod, hard rock, hole spacing close, steam.
9	Diorite (trap)	do	.2692	.54	3.05	2.36	Low face, benches, tripod, very hard rock, hole spacing close, steam.
13	Andesite (trap)	do	.4060	1.35	1.53	3.32	Low face, benches, tripods, very hard rock, some snake holes, spacing close, steam and electricity.
16	Rhyolite (trap)	do	.4358	2.18	.81	4.99	Low face, benches, tripods, very hard rock, spacing close, steam and electricity, steam and air.
2	Granite	do	.4482	.60	2.88	1.34	Low face, benches, tripods, very hard rock, spacing very close, steam and electricity, air.

[1] See Table 3 for spacing of holes.

[2] Snake holing with tripods also at this quarry cost $1.06 per foot, and averaged 1.34 feet per hour per drill.

[3] Undetermined.

should be useful in estimating on rock drilling. Table 2 gives data on the rate of drilling and the percentage of wear of the rock for the different kinds of drills.

TABLE 2.—*Relation between average feet drilled per hour by different types of drills and percentage of wear of rock as determined by laboratory test*

Tripod drills (vertical holes)			Tripod drills (snake holes)			Well drills		
Quarry key No.	Average feet drilled per hour	Per cent of wear	Quarry key No.	Average feet drilled per hour	Per cent of wear	Quarry key No.	Average feet drilled per hour	Per cent of wear
6	4.39	5.0	19	1.68	3.0	20	2.48	6.2
15	4.13	3.1	17	1.34	2.7	21	3.10	5.4
9	3.05	2.9	18	1.81	2.0	8	1.29	3.1
2	2.88	2.7				17	.65	2.7
12	2.34	3.0				7	.80	2.6
16	1.53	2.0						

At the time the studies were made little or no use was made of hammer drills in face work, although they were used in plugging bowlders. New types of hammer drills are now on the market and are widely used for bench work. They require only one man for operation and drill at a much faster rate than tripod drills. They are most successful when operated by air.

Blasting face.—Detailed data on blasting operations are shown in Table 3 in which the quarries are arranged in the order of the cost of blasting the face per ton. The highest costs were in connection with bench work in hard rock with few joints or with high faces in much jointed rock where snake holes were used. Some high costs resulted from wide spacing of vertical well-drill holes in high faces. Low costs would be expected in the large quarries but occasionally a small quarry is found in the low-cost group and a large quarry in the high-cost group.

The kind of rock, differences in joints (see Fig. 1), hole spacing and height of face were important factors affecting costs, but poor blasting often resulted in the need of further blasting for the removal of spurs and toes. Much blasting in small quarries was found to be haphazard or experimental.

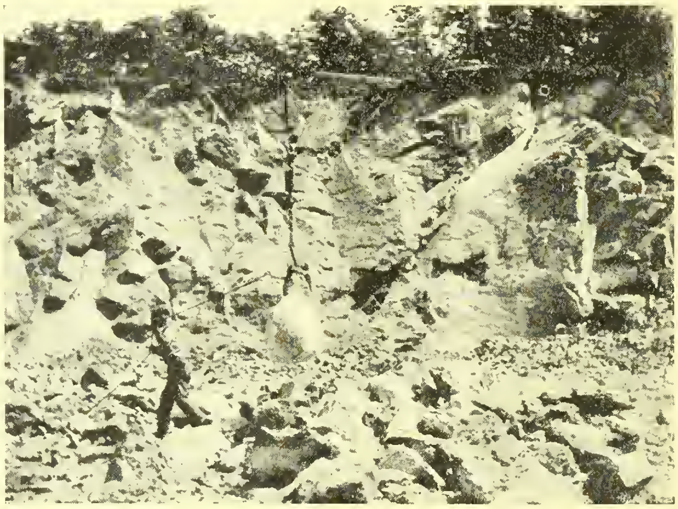


FIG. 2.—BREAKING BOWLDERS WITH A STEEL BALL

Analysis showed that the tonnage brought down per pound of dynamite is but slightly related to number and spacing of drill holes. In this respect there is a great difference between the operations of large and small producers. In the item of tonnage brought down per foot drilled, which is of much greater importance, the results were much more satisfactory in the quarries classified as large and very large from the production standpoint than in those classed as medium, small, and very small. There were conspicious exceptions to this, however.

It is concluded that there is a decided advantage in using deep, well-drill holes, widely spaced or, if the face is high and much jointed, snake holes should be used.

TABLE 3.—*Blasting face data, and direct costs arranged in order, from lowest to highest*

Quarry key No.	Direct cost of blasting face per ton of broken stone	Spacing of holes	Height of face blasted	Rock down per pound of dynamite	Rock down per foot drilled	Kind of rock and conditions
				Tons	*Tons*	
18	$0.0227	Snake holes 28 feet in	150 feet	12.9	69.09	Diabase, hard, very greatly jointed.
20	.0412	15 feet apart, 20 feet back	42 feet	4.9	24.53	Limestone, moderately jointed.
8	.0425	12 feet apart, 20 to 25 feet back	60 to 90 feet	5.8	27.24	Hard dolomite, strata slightly jointed, high face.
14	.0440	6 feet apart, 8 feet back	Benches, 18 to 20 feet	10.5	4.00	Rhyolite breccia, well jointed.
19	.0440	Snake holes 30 feet in	150 feet	6.6	32.25	Diabase, hard, highly jointed.
17	.0441	24 to 30 feet apart, 30 to 35 feet back	20 to 30 feet	6.7	[1] 53.47	Diabase, hard, many joints.
21	.0527	12 to 15 feet apart, 20 feet back	20 to 35 feet	3.9	19.23	Limestone, moderately jointed.
4	.0560	5 to 7 feet apart, 6 to 8 feet back	Benches, 14 to 18 feet	8.1	5.82	Granite moderately jointed.
12	.0574	3 to 4 feet apart, 18 feet back	Benches, 20 feet	8.0	5.19	Metamorphosed rhyolite, hard, many joints.
11	.0595	4 feet apart, 8 feet back	Benches, 14 feet	9.7	2.33	Hard conglomerate, well jointed.
5	.0609	5 to 6 feet apart, 15 feet back	Benches, 10 to 18 feet	7.3	2.53	Hard conglomerate, many joints.
7	.0616	12 feet apart, 20 to 25 feet back	70 to 90 feet	3.9	25.15	Hard dolomitic limestone, well jointed.
6	.0619	5 to 6 feet apart, 15 feet back	6 to 18 feet	6.2	2.45	Hard conglomerate, numerous joint planes.
16	.0630	4 feet apart, 6 to 8 feet back	Benches, 15 feet	5.4	4.99	Altered rhyolite, greatly jointed.
1	.0690	5 feet apart, 6 to 8 feet back	Benches, 20 feet	7.0	3.68	Hard granite and porphry, few joint planes.
3	.0794	10 feet apart, 20 feet back	60 feet	5.5	18.70	Granite, moderately jointed.
22	.0800	Snake holes 16 to 22 feet in	75 to 150 feet	6.0	8.51	Marble, hard, fairly jointed.
15	.0886	4 feet apart, 8 feet back	16 feet	5.1	2.31	Trachytic rhyolite, numerous incipient joints.
9	.0913	5 feet apart, 8 feet back	Benches, 20 feet	5.5	2.36	Hard and tough diorite, tight and few joints.
23	.0913	24 feet apart, 20 feet back	75 feet	2.2	24.09	Slate, very greatly jointed.
13	.0968	3 to 4 feet apart, 18 feet back	Benches, 18 to 20 feet	4.1	3.32	Altered andesite, hard, joints few and tight.
2	.1016	3 to 5 feet apart, 2 to 10 feet back	Benches, 20 feet	3.4	1.34	Hard granite, and porphry, few joint planes.
10	.1389	6 to 8 feet apart, 8 feet back	do	3.0	6.48	Diorite, hard and tough, relatively few joints.

[1] Includes well-drill holes and snake holes.

TABLE 4.—*Showing bowlder-breaking methods and costs per ton of broken stone produced*

Quarry key No.	Initial crusher size	Methods of breaking	Direct cost of drilling bowlders	Direct cost of blasting bowlders, mud capping, and block holing	Direct cost of steel balling bowlders	Direct cost of sledging bowlders	Direct cost of breaking bowlders, all methods	Average daily production in tons	Remarks
1	No. 5 gyratory	Block holing and sledging	$0.0714	$0.0827	None.	$0.3727	$0.5268	172	Granite, many bowlders, few fracture planes.
2	Two No. 5 gyratory independent units.	do	.1204	.0392	None.	.3149	.4745	260	Same quarry as key No. 1, following year, fewer bowlders due to change of method of blasting.
3	Jaw opening, 10 by 20 inches.	Steel balling and sledging	None.	None.	$0.0513	.0691	.1204	176	Well managed.
4	do	Mostly steel balling and sledging.	.0330	.0037	.0687	.1374	.2428	125	Same quarry as key No. 3, following year. Owner-manager sick, wages higher, labor much less efficient.
5	Two jaw crushers, 10 by 20 inches.	do	.0010	.0003	.2394	.2506	.4913	73	Very poor management and supervision.
6	No. 6 gyratory	do	.0003	.0002	.0175	.1517	.1697	292	Same as key No. 5, following year, good supervision.
7	Five No. 6 and one No. 4 gyratories.	Mostly mud capping; some block holing.	.0099	.0770	None.	None.	.0869	1,454	Large-scale operation, steam shovel to handle rock, large initial crushers.
8	Five No. 6 gyratories	Block holing and mud capping.	.0245	.1272	None.	None.	.1517	770	Do.
9	Jaw opening, 36 by 42 inches.	All block holing	.0514	.0184	None.	None.	.0698	234	Rock broke well on blasting face, fewer bowlders, large initial crusher, steam shovel.
10	No. 8 gyratory	Block holing and sledging	.0430	.0261	None.	.1627	.2318	532	Bowlders very tough but labor wasted in operation of sledging.
11	Jaw opening, 10 by 20 inches.	Steel balling, block holing	.0216	.0035	.1392	.2427	.4070	66	Inefficiency conspicuous, cost of steel balling and sledging much too high.
12	No. 6 gyratory	Steel balling and sledging	None.	.0009	.0487	.0939	.1435	214	Bowlders very tough.
13	Jaw opening, 16½ by 29 inches.	Block holing and sledging	.0284	.0201	None.	.0847	.1328	170	Numerous incipient fractures in bowlders, hence sledging cost low.
14	Jaw opening, 14 by 21 inches.	Mostly steel balling and sledging; some block holing.	.0196	.0024	.0945	.1804	.2969	82	Inefficiency, costs of steel balling and sledging too high.
15	do	Sledging	None.	None.	None.	.2501	.2501	86	No bowlder blasting permitted (city ordinance), hence every large bowlders broken by sledging, and cost of that item high.
16	Jaw opening, 36 by 18 inches.	Sledging and steel balling	None.	None.	.0073	.2063	.2136	180	Small bowlders hard to break, hence sledging cost high.
17	Jaw opening, 48 by 72 inches.	A little mud capping	None.	.0097	None.	None.	.0097	1,820	Large-scale plant, large initial crusher, steam shovel for loading, hence low costs.
18	No. 7½ gyratory	Very little bowlder breaking; mud capping.	.0047	.0113	None.	.0006	.0166	264	Rock broke well on blasting, relatively few bowlders, relatively large initial crusher, steam shovel for loading.
19	Jaw opening, 48 by 59 inches.	do	.0029	.0108	None.	None.	.0137	600	Do.
20	Jaw opening, 48 by 60 inches.	A little mud capping and block holing.	.0084	.0273	None.	None.	.0357	1,977	Conditions similar to those at key No. 17, see above.
21	Nos. 10, 9, and 8 gyratories.	do	.0093	.0171	None.	None.	.0264	1,772	Do.
22	No. 6 gyratory	Block holing and mud capping.	.0565	.1140	None.	.1351	.3056	176	Too many bowlders, due to blasting method.
23	No. 7½ gyratory	None	None.	None.	None.	None.	None.	94	Many joint planes, rock finely broken on blasting, hence no bowlders.

TABLE 5.—*Loading and delivery data*

Quarry key No.	Type of quarry	Average daily production in tons	Cost of loading per ton	Tons loaded per labor hour	Loading, common labor, rate per hour	Method of loading	Cost of delivery per ton	Cost per day, horse, cart, and driver
1	Open face	172	$0.1866	2.5	$0.45	By hand into high-sided tramcars	$0.0862	
2	do	260	.2056	2.5	.50	do	.1758	
3	do	176	.1235	4.6	.50	By hand into low-scale boxes resting on ground	.1283	
4	do	125	.0797	6.0	.50	do	.1347	
5	do	73	.2010	2.67	.50	By hand into high dump carts	.2071	$5.50
6	do	292	.1284	[1] 4.0	.50	By hand into low-scale boxes resting on ground	.0740	
7	Open face and pit	1,454	.1041	3.4	[2] .30	By steam shovel mostly, part by contract, hand labor	.0505	
8	Open face	770	.1667	3.2	[2] .30	By hand into tramcars, by contract, hand labor	.0667	
9	do	234	.1409		.39	Steam shovel to tramcar	.0328	
10	do	532	.0976		.44	Steam shovel and hand labor into low tramcars	.0382	
11	Pit	66	.2443	2.27	.50	Mostly by hand into high dump carts	.1251	[3] 4.50
12	Open face	214	.1521	3.05	.44	do	.1186	5.50
13	do	170	.2202	2.29	.45	do	.1301	5.00
14	do	82	.2040	2.7	.50	do	.1445	5.25
15	do	86	.3280	1.4	.42	do	.1784	5.40
16	Pit	180	.1788	[4] 2.9	.50	By hand into low-scale boxes resting on ground	.1459	
17	Open face	1,820	.0535		.40	Steam shovel	.0664	
18	do	264	.0902		.43	do	.0229	
19	do	600	.0729		.40	Electric shovel	.0376	
20	Pit	1,977	.0462		.50	Steam shovel	.0838	
21	Open face	1,772	.0538		.50	do	.0627	
22	Pit	176	.1294	3.5	[2] .50	By hand into tramcars, by contract, hand labor	.1514	
23	Open face	94	.1900	3.2	[2] .60	By hand into dump carts, contract labor	.0695	[5] 2.00

[1] Rate of tons per hour would have been still higher if more of the material loaded had not been unnecessarily handled twice.
[2] Calculated from contract basis.
[3] One driver for two 1-horse carts.
[4] Men lost time waiting for delivery to crusher.
[5] Cost of care and feed only; no driver employed.

Quarry key No.	Method of delivery	Average length of haul and grade	Power			Remarks
			Kind	Cost	Unit	
1	Cars pushed and switched by men, then hoisted up incline.	400 feet down grade for car pusher. Incline by hoist engine.	Coal	$10.00	Ton	Complicated method of delivery.
2	Two independent units to two crushers. Cars pushed and switched by men, then hoisted up incline.	do	do	10.75	do	Capacity of plant doubled by adding separate and independent unit, production actually increased only 50 per cent. Method of delivery complicated.
3	Derrick swing to pier; thence by car pusher down grade to crusher.	375 feet down grade	do	10.00	do	Loading and delivery methods excellent.
4	do	do	do	8.50	do	Do.
5	1-horse dump carts	450 feet upgrade	Men and horses			These methods always expensive.
6	By derrick direct to crusher	150 feet	Coal	9.50	Ton	Excellent system, cost would have been much lower but for unnecessary rehandling much of material during loading.
7	Steam locomotive mostly, part by mule to incline, then steam hoist.	800 feet, about level, small per cent up incline.	do	5.00	do	Typical of large-scale operations, these costs somewhat raised because of taking part of stone from a pit.
8	Men and mules on tram track.	500 feet level	Men and horses			Typical of large operations.
9	Cable pull by hoisting engine, automatic dump.	300 feet up gentle grade	Coal	10.00	Ton	One of the most efficient delivery systems, large, low tramcars, gentle upgrade, steam shovel expense too high, gravity return.
10	Horses and men	350 feet down grade	do	9.00	do	Good methods, well managed.
11	1-horse dump carts, also derrick hoist.	300 feet steep upgrade	do	10.00	do	Methods expensive, typical of most small-scale plants.
12	1-horse dump carts	200 feet upgrade	Men and horses			Do.
13	do	400 feet upgrade	do			Do.
14	do	225 feet upgrade	do			Do.
15	do	150 feet slight upgrade	do			Methods expensive, typical of most small-scale plants, men wasted time excessively.
16	By derrick and overhead cable system.	500 feet up	Coal	10.75	Ton	Costs could have been lower but for troubles with overhead cable system.
17	Steam locomotives	3,000 feet down grade	do	5.54	do	Typical of large-scale operations.
18	Car pusher, gravity	200 feet down grade	do	7.75	do	Power loading, gravity delivery, short distance and good management led to lower costs.
19	Gasoline locomotive	300 feet down grade	Gasoline Electricity	.24 .017	Gal Kwh	Typical of large-scale operations.
20	Steam locomotive to pit crusher, thence by hoist.	2,000 feet crusher in pit, then conveyor to main plant.	Coal	7.50	Ton	Do.
21	Steam locomotive	2,300 feet level	do	10.65	do	Do.
22	First by horse to foot of incline, second by electric hoist to crusher.	250 feet up by hoist engine.	Electricity	0.17	Kwh	Complicated method of delivery due to pit-type quarry.
23	First, by 1-horse carts to incline; second, by steam hoist to crusher.	125 feet level, then hoist engine to crusher in top of plant.	Coal	7.00	Ton	Loading done by contract labor at relatively higher price. Exceptional low cost for horses kept delivery costs low. Haul in quarry 125 feet, thence by bucket hoist 42 feet.

The 10 quarries with the lowest combined cost of drilling and blasting face used one of these two methods. Cost of the two items is not necessarily affected by production volume as the quarries ranking highest and lowest in this cost had about the same output.

STEEL BALLING THE MOST SATISFACTORY METHOD OF BOWLDER BREAKING

Bowlder breaking.—So many factors enter into bowlder-breaking costs that generalizations are difficult to make and the conditions at each quarry, as given in Table 4, must speak for themselves. The factors affecting the cost are efficiency of management, size of crusher opening, method of loading, drill-hole spacing and blasting, and the character of the rock itself. Illustrating the wide range in conditions it will be noted that at quarry No. 23 there was no bowlder breaking, while at No. 1 the cost amounted to more than 52 cents per ton of broken stone produced.

While it is difficult to give general conclusions, the studies do indicate that the breaking of bowlders by the steel-ball method can be used in many quarries to advantage. With this method (see fig. 2) one or more derricks operated by three-way hoisting engines are placed at a convenient distance from the face and by means of a chain and cable bowlders too large for sledging are dragged from the face and arranged beneath the derrick. The derrick is used to lift a steel ball weighing about 2 tons to a height of from 40 to 70 feet and drop it on the bowlder.

Great variation was observed in the success of this operation, but where skillfully directed, it was considered very successful. Comparing the cost of bowlder breaking at two small-scale quarries, where steel-balling was not used, with three as nearly similar quarries as it was possible to find where steel-balling as best practiced was used, it was found for the former that the cost of breaking bowlders per ton of broken stone averaged 26 cents while at the latter it averaged 14 cents.

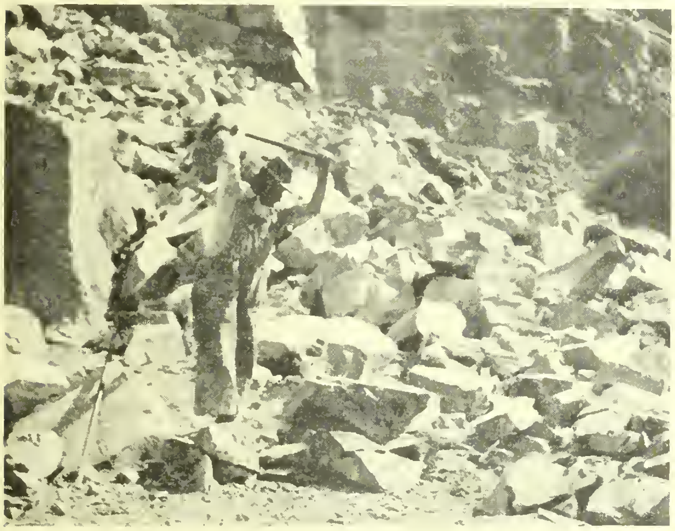


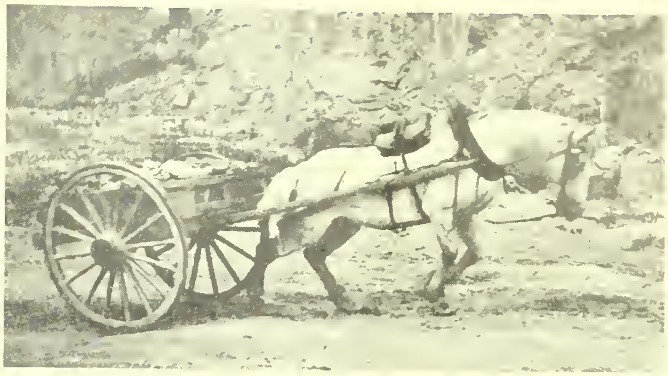
Fig. 3.—Sledging Is a Large Item of Expense at Most Small Quarries Where Rock Is Reduced by Hand Labor for Small Initial Crusher

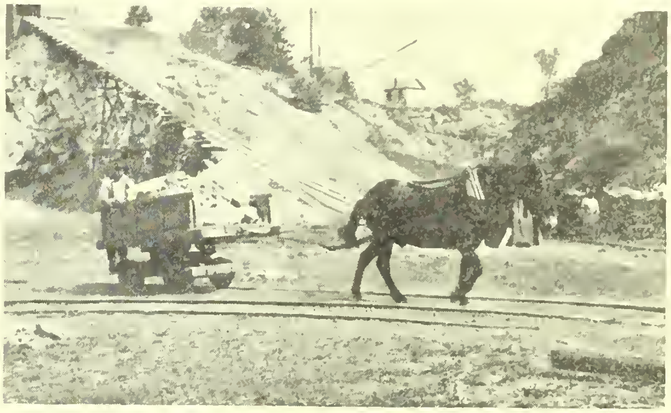
At medium and small-scale quarries the derricks used for steel-balling may also be used for transporting stone to the crusher and thus lower the cost of this item. If the distance is small, low scale boxes may be loaded and swung directly to the crusher, while if it is great they may be used for dumping into tram cars.

Pumping and disposal of waste.—These operations were so unusual that they did not enter materially into the production cost with one exception. In the pit-type quarries open seams permitted the escape of water except at one quarry where pumping cost $0.0256 per ton of broken stone. At only two quarries was it necessary to remove waste from the quarry floor. At one of these the cost was a fraction of a cent per ton of broken stone while at the other it amounted to 12 cents. At the latter quarry stripping often slid to the quarry floor and clay seams occurred between the beds of marble.

DUMP CART DELIVERY EXPENSIVE

Loading and delivery.—Loading and delivery have been considered together because a loading method must take into consideration the method of delivery.





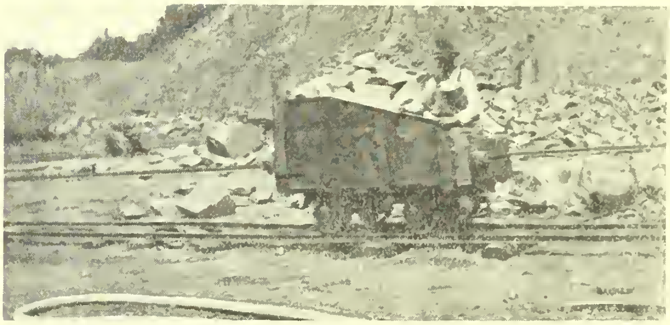


Fig. 4.—The Upper Picture Shows an Expensive Delivery Method, Which Cost 12½ Cents Per Ton. The Middle Picture Shows a Combination Method, Horses Drawing 2½-Ton Cars to the Incline and Thence by Steam Hoist. The Bottom Picture Shows an Economical Method, Coasting Cars Down Incline at a Cost of 3¾ Cents per Ton

Methods of delivery varied widely (see fig. 4) as would be expected in a series of plants selected because of varying conditions. Among the simplest methods were direct swing and hoist derricks, one-horse dump carts, tram cars pushed or coasted down grade to the crusher, direct cable haul to the crusher, and gasoline and steam locomotives. Numerous combinations of methods were found, most of which resulted from the problems of pit-type quarries. Table 5 shows the methods and costs of these operations at the various quarries.

In the item of loading it appears that many quarries could effect a considerable saving by a change in methods. It was found that a low receptacle could be loaded about twice as fast as a high one. (See fig. 5.) Five quarries using high dump carts loaded at a rate of 2.6 tons per hour per man, two using high tram cars loaded at a rate of 2.5 tons per hour, while two using low scale boxes loaded at a rate of 5 tons per hour. Assuming the same wage rate at all of the quarries the respective costs per ton were 21 cents, 20 cents, and 10 cents. Loading at five quarries using power shovels averaged 6 cents per ton, the range being from 4 to 9 cents.

Loading by contract labor was done at four quarries and was much more efficient than day labor. The contract labor averaged 3.3 tons per hour while the average for eight quarries using day labor was 2.6 tons per hour. In both groups the material was loaded into high tram cars and dump carts.

There was found to be a wide range in the cost of delivery, which demonstrates that some of the quarries were using poor methods, although some of the high rates could be attributed to adverse conditions.

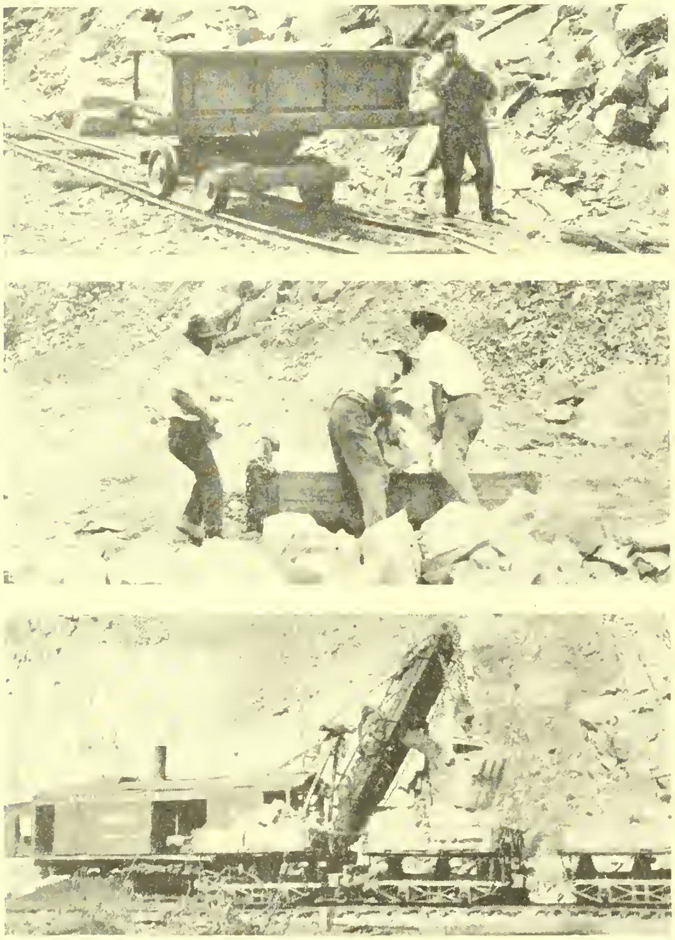


Fig. 5.—Height of Cart or Car Is an Important Factor in Loading in the Quarry Shown in the Upper Picture. Loading Was at a Rate of 2.4 Tons Per Hour, While in the Middle Picture the Rate Was 6 Tons Per Hour. The Bottom Picture Shows Large-Scale Steam-Shovel Loading

The lowest cost of delivery, $0.0229 per ton, resulted from coasting tramcars of 6-ton capacity an average distance of 200 feet down a gentle grade and returning the cars to the face by two men. The second lowest cost of $0.0328 per ton was incurred at a quarry where tramcars of 6-ton capacity were hauled to the crusher by a hoisting engine operating a cable a distance of 300 feet up a gentle grade which permitted a gravity return. Dumping was automatic. The third lowest cost resulted from gasoline-locomotive delivery of tramcars of 2-ton capacity a distance of 300 feet down a gentle grade. The average cost of delivery at six quarries by one-horse dump carts an average distance of 262 feet and in general slightly up grade was $0.1506, or about five times the average for the three quarries mentioned.

The average cost of delivery by steam locomotives over distances ranging from 2,300 to 3,000 feet was $0.0645 per ton. At pit-type quarries where steam locomotive hauls of 800 to 2,000 feet were combined with incline hoists the average cost per ton was $0.0672.

The use of the derricks used in steel balling for lifting and swinging scale boxes loaded with stone to the crusher platform has already been mentioned. At the quarry where this arrangement was used it resulted in a cost of $0.074 per ton and the cost of loading into the low scale boxes was also low. Figure 6 shows different methods of dumping at the crusher.

Crushing and screening.—Data on the cost of crushing and screening is presented in Table 6. Of the two quarries showing lowest costs, quarry No. 6 had an average daily production of only 292 tons of conglomerate rock which was easy to break, and quarry No. 21, 1,772 tons of limestone. Both used electric power. At quarry No. 6 a No. 6 gyratory crusher was used and very little crusher feeding was necessary.

Table 6.—*Crushing and screening data, and direct costs arranged in order, from lowest to highest*

Quarry key No.	Cost per ton of broken stone	Power elements		Average daily production in tons during period
		Kind	Cost per unit	
6	$0.0493	Electricity	$0.027 per kwh	292
21	.0630	do	$0.018 per kwh	1,772
19	.0724	do	$0.017 per kwh	600
18	.0764	do	$0.030 per kwh	264
16	.0816	Coal	$10.75 per ton	180
10	.0823	Electricity	$0.027 per kwh	532
17	.0827	do	$0.015 per kwh	1,820
7	.0836	do	$0.009 per kwh	1,454
2	.0896	Coal Electricity	$10.75 per ton $0.027 per kwh	260
15	.0904	Electricity	$0.030 per kwh	86
20	.1013	Coal	$7.50 per ton	1,977
13	.1060	Electricity	$0.027 per kwh	170
8	.1069	do	$0.009 per kwh	770
12	.1103	do	$0.027 per kwh	214
4	.1170	do	$0.027 per kwh	125
22	.1215	do	$0.017 per kwh	176
3	.1271	do	$0.027 per kwh	176
14	.1408	do	$0.027 per kwh	82
11	.1437	do	$0.027 per kwh	66
23	.1569	Coal	$7 per ton	94
9	.1737	do	$10 per ton	234
1	.2052	do	$10 per ton	172
5	.3423	do	$11.75 per ton	73

Two of the quarries listed were under the same management and crushing the same kind of rock with modern plants. At one of these quarries steam was used for power, at the other electricity. Crushing with steam power cost $0.1013 per ton while crushing with electric power cost only $0.0630 per ton. This difference in cost is believed to be largely the difference in cost of power.

GENERAL CONCLUSIONS

Table 7 shows the total direct cost per ton for each of the quarries with information as to the kind and size of quarry. Comparisons of all the quarries as to cost of the various operations show that none of them maintained a uniform high position on all items. There is a general tendency for a quarry to be among the highest group with regard to one operation and much lower down the scale with regard to the next. The total direct costs do in general vary inversely with production, but the difference would be less conspicuous if these costs included rental, or interest and depreciation on power equipment. It is believed that a relatively small-scale quarry can be operated with direct costs lower than the average, or even the best record of the large-scale quarries if good management is combined with a good layout, good equipment, electric power, and favorable conditions of face and rock. Results at quarry No. 18 demonstrate this, as it had the lowest direct production cost of any of the 23 quarries and yet had an average daily production of but 264 tons.

Table 7.—*Total direct production costs, kind of rock, magnitude of operation, and price of labor per hour*

Quarry key No.	Direct production cost per ton	Kind of rock	Magnitude of operation	Price of labor per hour
1	$1.3179	Granite	Small	$0.45
2	1.5250	do	Medium	.50
3	.6606	Altered granite	Small	.50
4	.8987	do	do	.50
5	1.5648	Conglomerate	Very small	.50
6	.7328	do	Medium	.50
7	.4242	Siliceous dolomite	Very large	.30
8	.5595	Dolomite	Large	.30
9	.7931	Altered diorite	Medium	.39
10	.7077	Diorite	Large	.44
11	1.2503	Altered rhyolite	Very small	.50
12	.7261	do	Medium	.44
13	1.1095	Altered andesite	Small	.15
14	1.0684	Rhyolite breccia	Very small	.50
15	1,1547	Trachytic rhyolite	do	.42
16	1.1187	Altered rhyolite	Medium	.50
17	.2828	Altered diabase	Very large	.40
18	.2535	Diabase	Medium	.43
19	.2962	Altered diabase	Large	.40
20	.3471	Dolomite limestone	Very large	.50
21	.3130	Limestone	do	.50
22	.9937	Marble	Small	.50
23	.6546	Siliceous slate	Very small	.39

At many plants, especially the smaller ones, only a crude system of bookkeeping is used. As a rule the distinction is not made between bookkeeping and cost keeping. If total costs are too high, the owner does not know where the trouble lies. Indeed, within a wide range of costs that lie inside the profit line, he is likely not to know whether any or all of his costs are too high or what methods, if any, should be changed, or whether a foreman or superintendent is getting the best possible results from labor. As this is true with direct costs so it is true with indirect costs, and a very common fault is to take little or no account of such important factors as depreciation and depletion.

The writer studied costs at three quarries located in the suburbs of a large city which were owned and operated by a contractor doing a large business. Most of the broken stone produced at these quarries was used on his contract jobs. Such stone as was sold was sold by the yard and measured in the delivery trucks. All the rest of his product was sent by truck to centrally located scales for weighing. Incredible as it may seem no record was made as to which of the three quarries the stone came from and no individual records were kept of their output. The owner only knew the total production of the three quarries which were several miles apart and under separate managements. As a matter of fact one of them was very badly managed and direct costs were about as high as they could be made.

With reference to face breaking, there is no question but that modern types of hammer drills should be used for bench work at least, and that in the long run the use of air instead of steam is economical in spite of the necessary investment in a compressor. Well drilling and wide spacing of holes and snake holing high faces reduce very largely the total costs of face breaking, and the type of explosive used should be determined by both experiment and advice from manufacturers.

Where the scale of operations is not too large the most satisfactory method of bowlder breaking is steel balling, although this might not be applicable to the breaking of bowlders in some of the tough diabases which are sometimes quarried. This must be determined experimentally. Comparative results in these studies indicate that mud capping is the most expensive method.

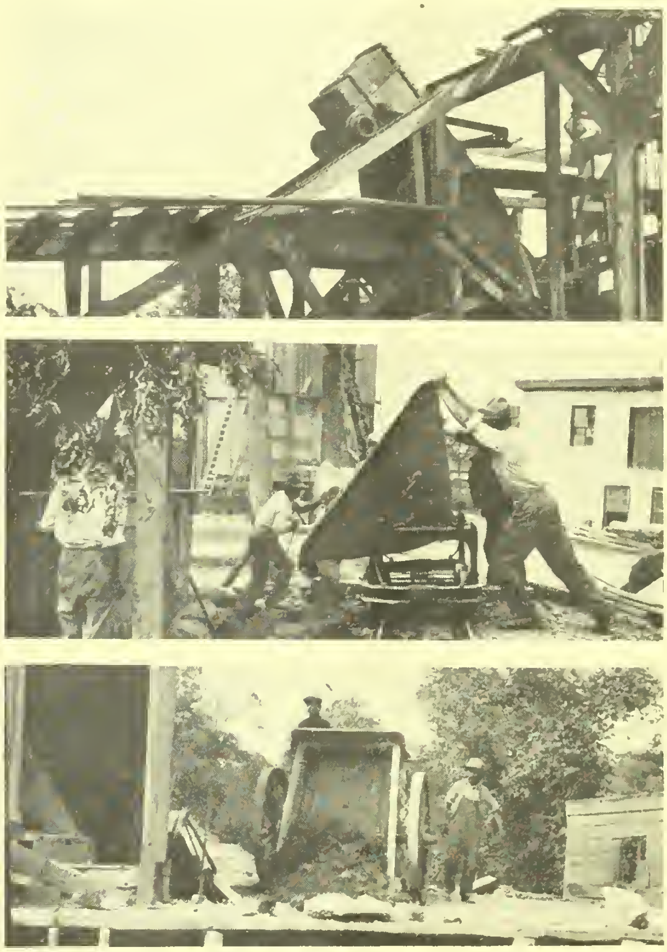


Fig. 6.—The Top Picture Shows an Arrangement for Automatic Dumping of 5-Ton Tram Cars, the Middle Picture Shows a Good Hand-Dump Tram Car, and the Lower Picture Shows a Cart Dumping on Crusher Platform, Requiring About Half of Stone to Be Hand Fed

Loading by hand into high receptacles costs from two to three times as much as loading into low-scale boxes. Steam shovels may be economically operated in comparatively small quarries and a great deal of sledging expense saved if a reasonably large initial crusher is used.

The horse-drawn dump cart is most expensive as to both loading and delivery. Delivery by derrick, directly, and by tramcars into which the contents of derrick-handled scale boxes are dumped, are relatively inexpensive, and the results as shown in costs of delivery at quarries Nos. 3, 4, and 6 do not do justice to the method, because in each case with insignificant increase of expense the number of tons delivered could have been more than doubled, while the results by the dump-cart method were for the maximum capacity of operations, and any increase in capacity called for a proportional increase in expense.

For distances up to 300 feet at least delivery is very economically accomplished, if grade permits it, by hand-pushed or horse-drawn tramcars of 2 to 4 tons capacity. Of all the methods of delivery studied the cheapest was the method using automatic dumping, 6-ton tramcars (steam-shovel loaded) pulled up a gentle grade to the crusher by a drum hoist and returned to the quarry face by gravity. A few posts set in the quarry floor as cable guides made it possible to draw these tramcars around curves, and thus made all parts of the face available for this method without unnecessary track complications or extra assistance other than such as could be given as needed by the quarry foreman. Power locomotives are of course necessary for long hauls and large operations, and simple arrangements cut down delivery expense.

Crushers above quarry floors and complicated track and switch arrangements result in high cost of delivery to the crusher. Pit-type quarries, as would be expected, have relatively high costs for delivery.

The kind of power used has much to do with costs. The more extensively electricity is used the more economically are most of the operations conducted.

At most small-scale quarries the initial crusher is too small. The saving through use of a small crusher is usually lost several times over by increased sledging costs.

For large-scale operations plants are usually designed, methods adopted, and layouts planned by competent engineers. On the other hand, for small plants these features usually are the result of expediency, experiment, or guesswork. Sometimes they result from previous experiences, good or bad, where very different conditions prevailed. As a rule insufficient attention is paid to the relations between the cost of bowlder breaking and the methods of loading, drilling, blasting, and the size of initial crusher. The method of one operation is dependent for success upon that of another. In other words, operation methods are closely interdependent.

Whenever possible in establishing a broken-stone plant the probable duration and both present and probable future magnitude of operations should be first determined. A large plant is, of course, not justified by small ledge holdings, and the relation between these two must be recognized. Topography must be studied and, if possible, a site for the plant procured below the level selected for the quarry floor and as near the face as blasting operations will safely permit. The ledge itself should be studied with reference to joint planes and seams, and the rock itself tested for percentage of wear. Conditions as to the former will indicate drill-hole spacing and blasting methods likely to be most satisfactory, and the latter will give information as to the probable cost of drilling.

If these considerations, together with the foregoing notes on various operations, are kept in mind they should lead to production costs lower than those now incurred at very many existing plants.

Finally it should be stated that no plant can afford to omit the keeping of costs on each unit of operation.

(Continued from p. 162)

PRELIMINARY AND FINAL SOIL CLASSIFICATION

Suppose we get a shipment of soil samples from a certain section of the country and we face the problem of comparing the soils with those previously received from other districts. Since an exhaustive investigation of every one of the samples would be financially forbidding, we have to start with a rapid survey of the material for the purpose of separating the soils into groups with fairly similar properties. This survey consists in determining for each soil the limits and in representing the limits in a regional soil diagram of the type of Figures 8 to 12 and Figure 14. The next step consists in classifying the soils according to their lower liquid limit.

For material with a low liquid limit, a difference of 5 per cent between the liquid limits has obviously more weight than the same difference for materials with a very high liquid limit. Hence the classification according to the liquid limit is made as follows:

Class	Liquid limit per cent
0	10 to $10\times 2^{1/2}=10$ to 14.2
I	$10\times 2^{1/2}$ to $10\times 2=14.2$ to 20.0.
II	10×2 to $10\times 2^{3/2}=20.0$ to 28.4.
III	$10\times 2^{3/2}$ to $10\times 2^{2}=28.4$ to 40.0.
IV	10×2^{2} to $10\times 2^{5/2}=40.0$ to 56.8.
V	$10\times 2^{5/2}$ to $10\times 2^{3}=56.8$ to 80.0.
VI	10×2^{3} to $10\times 2^{7/2}=80.0$ to 113.6.

If we have found, for instance, that a soil belongs in the class V, indicating a very high liquid limit, we know that the soil may either be excessively fine grained or else very rich in flexible, scale-like particles. The soil will be very different according to whether the first or the second assumption is correct. Hence, within each one of these groups distinction is made between soils with a low, medium or high plasticity index. The occurrence of a high liquid limit with a low plasticity index is very probably due to excessive fineness of the soil, while another soil with a high index is very probably distinguished by an abundance of scale-like particles which in turn seems to be associated with a very marked tendency to swell in contact with water. The shrinkage limit serves as an additional check on the degree of relationship between the members of each group.

However, a final classification should not be based on data as vague as the conclusions we can draw from the limits. Hence, in order to establish a permanent foundation for soil classification, we have to go one step further and investigate what the moduli of compression, of expansion, and the coefficients of permeability of the members of the different groups are. Since these moduli depend on precisely the same factors as the limits, there must be a numerical relationship between them and the knowledge of this relationship will ultimately bring the interpretation of the limits to a certain state of perfection. However, the classification itself will have to be based on the numerical values of the three characteristic constants, viz, the modulus of compression, the modulus of expansion, and the coefficient of permeability at a standard pressure, because these are the only numerical values which have a direct bearing on the mechanical properties of the subgrade, including its resistance.

Any well organized soils laboratory will ultimately arrive at empirical rules for translating the results of its own limit determinations into physical terms. But, on the other hand, it is very doubtful whether the same rules will ever apply to limits determined by different laboratories. In spite of carefully standardized instructions, a certain personal element inevitably adheres to every limit determination and, for this reason, the necessity for checking the limits against the results of well defined physical tests will always exist.

CONCRETE COMPARED WITH TIMBER FOR HIGHWAY BRIDGE FLOORS

Reported by **O. L. GROVER**, Bridge Engineer, Bureau of Public Roads

THE RENEWAL of plank floors on bridges is a large item in the annual expense of bridge maintenance; and aside from its expense, which is heavy, wooden flooring is noisy and causes excessive vibration of the structure and objectionable jolting of traffic when improperly laid or in a worn condition. While treated timber is expected to resist decay for about 25 years, unprotected flooring has a service life of from 3 to 6 years only, depending on the amount and character of traffic; and on bridges which carry a large volume of traffic or heavy units, such a floor wears out so fast that maintenance is difficult and the traffic is more or less unsatisfactorily served at all times in spite of heavy outlays for repairs.

Figures obtained by averaging the cost of maintaining unprotected timber floors on several steel bridges over a number of years show what the cost of upkeep may amount to on bridges carrying main-route traffic. In the District of Columbia for a 30-year period prior to 1926 the average cost for repair and renewal was about 3½ cents per square foot per annum, or about 70 cents per lineal foot for a 20-foot roadway, while for a 10-year period prior to 1926 the average was higher chiefly because of changes in the amount and character of traffic. In general it may be said that the experience of the past 10 years indicates a cost of about 6 cents per square foot per annum, or $1.20 per lineal foot for maintaining a 20-foot floor on bridges carrying present traffic on through highways near cities.

Such costs are excessive. In the effort to reduce them and improve the service of the unprotected floor, where the strength of the bridge has been adequate to carry the added dead load, bituminous surfacing has been tried as one method. Another which has met with some favor is the addition of steel-plate traffic treads; a third is the addition of a longitudinal or diagonal layer of timber as a wearing surface; and the use of planed flooring instead of rough plank has also been given a reasonable trial. The object of the latter method is to reduce wear by building a smooth floor, and this is accomplished by the uniform thickness of the planed flooring. The beneficial effect of this method, however, is gradually lost by reason of the wear and replacement of occasional pieces with new pieces, the depth of which naturally varies considerably from that of the adjacent flooring.

For a 20-foot roadway the average unit prices of these are approximately as follows:

Bituminous surfacing 2 inches thick, per lineal foot	$5. 55
Four lines of steel tread, 24 inches wide by $\frac{5}{16}$ inch thick, per lineal foot	8. 96
Four lines of steel tread, 24 inches wide by $\frac{3}{16}$ inch thick, per lineal foot	5. 60

It thus appears that the 2-inch bituminous surfacing costs $3.41 less per lineal foot than the 5/16-inch treads and 5 cents less than the 3/16-inch treads. But these comparisons are naturally affected by the relative prices of steel and bituminous materials, and since the former vary from about 7 to 11 cents a pound and the latter from approximately $1.75 to $3.50 a square yard the relative cost of the two methods may vary considerably from the above averages which are based on steel at 8 cents and bituminous surfacing at $2.25 a square yard. Whichever, under the particular conditions obtaining, may be the cheaper, it is probable that the reduction in the cost of maintaining the floor will be more than sufficient to pay for the covering. But the resulting construction is still open to some of the same objections which pertain to unprotected plank floors, and some of these are serious.

With these objections in mind Dr. J. A. L. Waddell declared in 1919: "Wooden floors on highway bridges are now obsolete," adding that "no modern bridge engineer should be guilty of designing or building any more of them." He gave as his reasons for these very positive statements the following three reasons:

"First. The ordinary plank floor supported on wooden joists is not strong enough to carry with safety the heavy, concentrated live loads produced by rapidly moving auto trucks.

"Second. The danger from fire is so great that it does not pay to submit the entire construction to the chance of destruction by the burning of the timber in the deck.

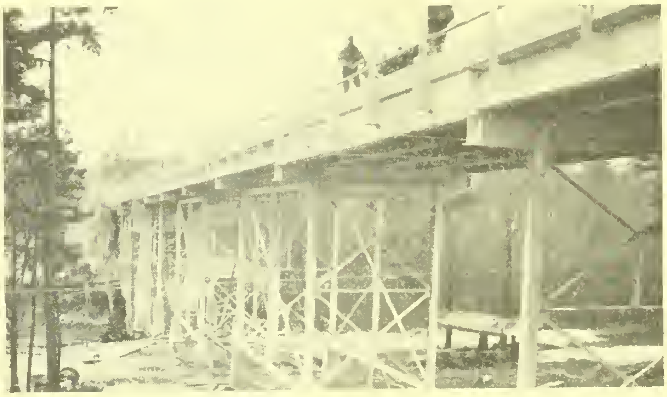


Fig. 1.—Steel Viaduct With Concrete Floor Carried on Concrete Pedestals. View Shows the Span Over Railroad Track Incased in Concrete

"Third. The upkeep of a wooden floor is far higher than that of a paved floor on a concrete base; and the difference in the future will steadily increase, because the cost of timber will rise with its augmenting scarcity."

COSTS OF CONCRETE AND TIMBER FLOORS COMPARED

General adoption of the concrete floor, for which Doctor Waddell so emphatically declared seven years ago, has been retarded to a certain extent by the unsatisfactory experience with the older type, used extensively in and near cities, consisting of a concrete bed for block or bituminous surfacing carried by steel buckle plates. This was the type used before the adoption of reinforced concrete slabs, and it has proved to be expensive both to construct and to maintain. The defects usually noted are broken buckle plates, especially where the buckle is turned up, broken and loose concrete, and frequent expensive repairs resulting from the breaking up of the pavement and holes in the floor.

The reinforced slab design is not open to these objections; and in common with the older type it possesses the very desirable quality of rigidity which inspires the confidence of those who ride over it. To the engineer it also suggests the idea of heavier weight, and the first thought is likely to be that extra steel will be required to carry it, so much, perhaps, that the cost of the bridge with concrete floor may exceed the cost of a timber-floored structure. So far as the trusses and floor beams are concerned this assumption is correct, but on account of the possibility of using deeper I beams for stringers and a wider spacing the saving in stringer weight may be nearly enough to compensate for the increased weight of trusses and beams.

To show the effect of using a concrete instead of a wooden floor upon the cost of actual structures of various types a number of cases are cited below in which comparative estimates or alternate bids on the two types of construction are available.

Case 1.—The structure shown in Figure 1 consists of a steel viaduct on concrete pedestals and open abutment bents with riprapped embankment around the ends of the fills. It crosses a railroad track and a creek about 70 feet wide. The entire structure is 379 feet in length, with span lengths as follows:

	Feet
1 deck truss	65
2 deck I-beam spans, each 40 feet	80
5 deck I-beam spans, each 32 feet	160
2 deck I-beam spans, each 37 feet	74

The roadway width is 20 feet.

For this structure alternate designs were made, one with a concrete, the other with a timber floor, both to carry a concentrated live load of two 15-ton trucks with 67 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact.

While the preliminary estimates indicated that the cost of the structure would probably be increased 2½ per cent by the concrete floor, the contract was let at no increase. One of the 40-foot I-beam spans over the railroad track, as constructed, has its beams encased in concrete. This was not provided for in the alternate timber-floor scheme. The estimate of the cost of the concrete-floored structure complete, based on bid prices and including 10 per cent for incidentals, is $44,000.

Case 2.—Another bridge, now under construction, crosses a river the bed of which is erodible to a considerable depth. It is designed to carry two 15-ton trucks with 67 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact.

Alternate designs for concrete and timber-floored structures were made for two arrangements of spans including varying span lengths. For one arrangement the estimate of the cost of the concrete-floored structure was about 2 per cent higher than that for the wooden-floored design; for the other arrangement the concrete floor caused an increase of about 4 per cent in cost.

The first arrangement with concrete floor and a roadway width of 18 feet is now under construction. It consists of three steel spans between piers 130, 190, and 320 feet apart, and eight timber trestle spans, each 19 feet in length, making a total length of 792 feet. The contract price for this structure, exclusive of approaches, is $180,000.

COST INCREASED ONLY 2 PER CENT BY CHANGE FROM WOOD TO CONCRETE

Case 3.—In another case which recently came to the writer's attention a design was made for two spans with an 18-foot roadway, one 180 and the other 200 feet in length, with a concrete floor, after preliminary designs had been made for the same spans with a timber floor.

In each case the design was made for a concentrated live load of two 15-ton trucks with 80 per cent of their weight on the rear axle, plus an allowance of 30 per cent of the live load stress for impact.

For the entire structure the estimates showed a margin in favor of the wooden-floored design amounting to $617, the cost with concrete floor being $33,425. For the two spans the difference was thus shown to be about $1.63 per foot or less than 2 per cent of the total cost of the wooden-floored structure. For the 180-foot span the difference was $1.58 per foot; for the 200-foot span about $1.67 per foot. The comparative estimates for the two designs of each span were as follows:

Comparative estimates of cost of the 180-foot span

1. WITH CONCRETE FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Item	Quantity	Unit price	Total
Concrete	88 cubic yards	$22.00	$1,936.00
Structural steel	214,000 pounds	.06	12,840.00
Reinforcing steel	12,500 pounds	.055	687.50
Total			15,463.50

2. WITH CREOSOTED TIMBER FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Structural steel	210,000 pounds	$0.06	$12,600.00
Creosoted timber	25,800 feet b. m	.10	2,580.00
Total			15,180.00

Comparative estimates of cost of the 200-foot span

1. WITH CONCRETE FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Item	Quantity	Unit price	Total
Concrete	98.0 cubic yards	$22.00	$2,156.00
Structural steel	250,500 pounds	.06	15,030.00
Reinforcing steel	14,100 pounds	.055	775.50
Total			17,961.50

2. WITH CREOSOTED TIMBER FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Structural steel	246,300 pounds	$0.06	$14,778.00
Creosoted timber	28,500 feet b. m.	.10	2,850.00
Total			17,628.00

Case 4.—A bridge which is about one year old includes three 100-foot spans with a roadway of 18 feet. It has a creosoted timber floor with four steel-plate traffic treads 24 inches wide by 3/16 inch thick. The actual cost of this structure at the prices bid for the several items was $8,362.49 for each 100-foot span as shown in the tabulation below.

Recently a bridge of the same dimensions, but with a concrete instead of a timber floor, was designed for construction on a forest road. In each case the design was made for a concentrated live load of two 15-ton trucks with 80 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact. Using the price bid for structural steel in the older bridge and reasonable prices for concrete and reinforcing steel it is estimated that the forest road bridge with concrete floor can be built at a price of $7,906.01 for each 100-foot span as shown in detail below.

Comparison of these designs and the estimates below indicates that it probably would have been possible to substitute for the older design with wooden floor economically designed steel spans with concrete floor at a cost materially less than the price paid for the actual timber-floored structure with steel traffic treads.

The actual cost of the wooden-floored bridge per 100-foot span and the estimated cost of the equivalent structure with concrete floor are as follows:

1. ACTUAL COST OF 100-FOOT SPAN WITH 18-FOOT TIMBER FLOOR AND 4 STEEL-PLATE TRAFFIC TREADS

Item	Quantity	Unit price	Total
Structural steel	96,700 pounds	$0.0687	$6,643.29
Creosoted timber	10,400 feet b. m.	.098	1,019.20
Traffic treads	7,000 pounds	.10	700.00
Total			8,362.49

2. ESTIMATED COST OF 100-FOOT SPAN WITH 18-FOOT CONCRETE FLOOR

Structural steel	92,300 pounds	$0.0687	$6,341.01
Reinforcing steel	6,800 pounds	.05	340.00
Concrete	49 cubic yards	25.00	1,225.00
Total			7,906.01

BITUMINOUS SURFACED CONCRETE COSTS LESS THAN WOOD WITH HEAVY TRAFFIC TREADS

Case 5.—Two designs were recently prepared for the same span and roadway, 135 feet and 20 feet respectively. Each provided for a concentrated live road of two 15-ton trucks with 80 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact. One was made for a concrete floor, the other for a creosoted timber floor, each to carry 2 inches of bituminous surfacing.

As shown by the following estimates of the cost of the two designs the one with a concrete floor may be expected to cost $165 more than the wooden-floor design, which is $1.22 per lineal foot and only about 1¼ per cent above the cost of the wooden-floored structure.

Comparative estimates of cost of the two designs

1. WITH CONCRETE FLOOR AND 2-INCH BITUMINOUS SURFACE

Item	Quantity	Unit price	Total
Structural steel	148,600 pounds	$0.069	$10,253.40
Reinforcing steel	9,300 pounds	.055	511.50
Concrete	72 cubic yards	25.00	1,800.00
Bituminous surface	292 square yards	2.50	730.00
Total			13,294.90

2. WITH CREOSOTED TIMBER FLOOR AND 2-INCH BITUMINOUS SURFACE

Structural steel	153,500 pounds	$0.069	$10,591.50
Creosoted timber	16,440 feet B. M	.11	1,808.40
Bituminous surface	292 square yards	2.50	730.00
Total			13,129.90

With four lines of steel-plate traffic treads, weighing 9,400 pounds, estimated at the low cost of 8 cents a pound the cost of this structure would be $13,151.90, or only $143 less than the structure with a concrete floor and bituminous surfacing. This estimate is based on the use of 24-inch by 3/16-inch treads which is the usual practice, although it is doubtful if plates so thin will give good service for a long time, especially if the corners of the plates are not bolted down with through bolts. If $\frac{5}{16}$-inch plates are assumed, the estimated cost becomes $13,609.50, which is $314.60 higher than the cost of the bridge with concrete floor and bituminous surfacing, a difference of more than 2 per cent of the total cost in favor of the concrete-floored structure. Without the bituminous surfacing the concrete-floored design would cost only $12,564.90 and would then have a favorable margin of $587 compared with the design with timber floor and thin traffic treads, a difference amounting to more than 4 per cent of the total cost.

OTHER ADVANTAGES OF CONCRETE

It would seem that the above comparisons fully support the declaration by Doctor Waddell in favor of concrete floors from the standpoint of economy. From other points of view it would also seem that there might be added to the three reasons he enumerates, as the basis for his preference, several others, as follows:

1. That a well-shaped and watertight concrete floor so formed as to conduct the surface water to outlets away from the steel work gives better protection from the corrosive effects of weather and dirt to the steel under it.
2. That it is maintained with less labor and that its upkeep, therefore, causes less interference with traffic.
3. That the first cost of a concrete-floored bridge designed to carry heavy truck loads is generally but little more than a timber-floored design for the same live load Even where prices are unfavorable to concrete the cost may only slightly exceed that of a timber-floored design.
4. A concrete floor increases the rigidity of the spans and in this respect acts as an aid to the bracing.

C. S. JARVIS IS AWARDED MEDAL FOR PAPER ON FLOOD FLOW

C. S. Jarvis, associate highway engineer of the Bureau of Public Roads, has been awarded the J. James R. Croes medal for his paper "Flood-Flow Characteristics" by the American Society of Civil Engineers. This medal is considered as next to the highest honor of this character conferred by the society and is awarded for a paper which is judged worthy of special commendation for its merit as a contribution to engineering science.

Mr. Jarvis has for several years made a special study of flood flow in streams in addition to his regular highway engineering duties. His object has been to arrive at some method whereby the maximum flood flow of a stream can be determined with a fair degree of accuracy from such information as can be collected in the course of a survey for a bridge or river control work.

THE STRENGTH OF MORTAR AND CONCRETE AS INFLUENCED BY THE GRADING OF THE SAND

By T. C. POWERS, Chemist, Department of Materials, Oregon State Highway Commission

IN AN article under the above title appearing in the July number of Public Roads the following four conclusions are reached:

"1. That there is an ideal grading of sands which will produce maximum strength in concrete.

"2. That the ideal grading curve assumes an arched form showing a predominance of the material retained upon the coarser sieves.

"3. That for a given mix, there is a practical limit to the quantity of material passing each size sieve, where a given strength of concrete is required.

"4. That an exceptionally high tensile strength of sand in 1: 3 mortar is not necessarily associated with a high compression strength of the same material when mixed with the average coarse aggregate in concrete; hence the tensile strength is not a proper gauge of the quality of a sand for concrete."

In this article the author discusses the paper published under the same title in the July issue of PUBLIC ROADS from the point of view of the fineness modulus theory and offers an interpretation of the data presented in the original article which he believes to be more in accord with the fundamentals of concrete designing.

He then presents, as a sequel to the original paper and his discussion, a statement of his belief as to what is really wrong with the 1: 3 mortar test.

It was found that for a 1: 2: 4 mix the grading giving maximum strength of concrete in compression was not the same as the grading giving maximum strength in tension in a 1: 3 mortar, and from this fact the conclusion is quite naturally drawn that tensile strength is not a proper gauge of the quality of a sand. However, to the writer, the most important thing illustrated by this investigation is the absurdity of proportioning concrete by an arbitrary mix whenever a given strength is sought. To show this the writer wishes to offer another interpretation of the sand analysis chart published with the article.

Let us consider first the middle ("ideal") curve. While the article does not so state, it is assumed that, since the object was to investigate sand, the coarse aggregate was the same or very similar in all tests.[1] Let us assume, then, that the coarse aggregate has a fineness modulus of 7.5 and a maximum size of 2 inches. (At least 15 per cent retained on the 1½-inch sieve.) From the curve, the ideal sand shows a fineness modulus of about 3.30. Then in a 1: 2: 4 mix the combined aggregate would have a fineness modulus of 6.16, which is about the maximum permissible fineness modulus for the above proportion of cement.[2] To increase the fineness modulus beyond this value would result in an aggregate too coarse for the amount of cement used, and a decrease in strength would result. Furthermore, to decrease the fineness modulus would cause an increase in mixing water for a given consistency with a corresponding decrease in strength.

With this in mind let us examine the top curve of the chart, which is reproduced herewith as Figure 1. The fineness modulus is about 4.00; just about the maximum permissible value for a 1:3 mix with an aggregate graded from zero to four. But, in a 1: 2: 4 mix this sand combined with the same coarse aggregate as before would result in a fineness modulus of 6.40 for the combined aggregate. This is too coarse for the amount of cement used and the strength of the concrete is lower. To equal the strength of the concrete developed by the "ideal" curve the mix would have to be made about one part cement to four parts combined aggregate.

The fineness modulus of the bottom curve is about 2.80. This, combined with the coarse aggregate in a 1: 2: 4 mix would give the combined aggregate fineness modulus of 5.92, which would require more mixing water than the 6.16 of the middle curve. A lower strength is the result.

From the foregoing it should be clear that there is no significance in the fact that the tensile strength of the 1:3 mortar did not reach a maximum as the coarseness of the sand increased, while the compressive strength of the 1:2:4 concrete did. The reason is that the sand did not become too coarse for the 1:3 mortar, but the combined aggregate did become too coarse for a 1:2:4 mix. See Figures 2 and 3.

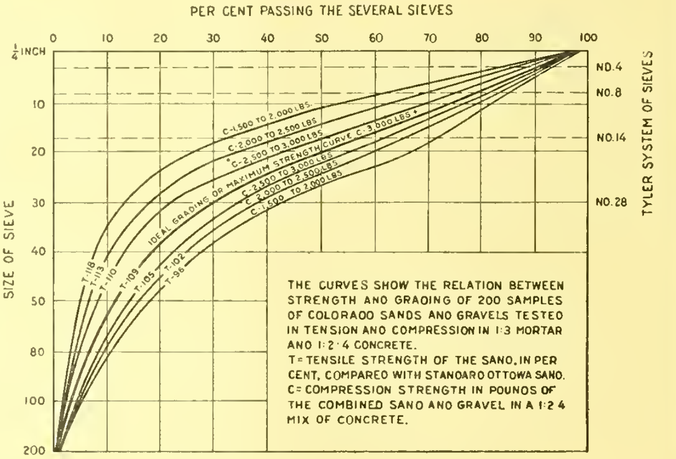


Fig. 1.—Sand Analysis Chart

ANOTHER VIEW OF WHAT THE GRAPH SHOWS

It seems, therefore, that the conclusion that ". . . tensile strength is not a proper gauge of the quality of a sand for concrete" is not justified by the data presented. If the finer or coarser sand were properly combined, the same strength as the "ideal" could be obtained without increasing the cement, all other factors such as structure and cleanliness being equal. Let us illustrate by again taking the top and bottom curves for consideration.

By changing the mix to 1: 2.4: 3.6, using the sand represented by the top curve, the combined aggregate would have a fineness modulus of 6.16, the same as the modulus obtained by using the "ideal" sand. All else being equal, the strength would be the same as the ideal of the chart.

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[1] This assumption is not correct. The coarse aggregate was not the same in all tests. Mr. Rose's study was based upon the results of laboratory tests by the State of Colorado, and it is known that the coarse aggregates represented were not uniform.

[2] See Bul. No. 1, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

By changing the mix to 1:8.8:4.2, using the sand represented by the bottom curve, the combined aggregate would have a fineness modulus of 6.16 and again, all else being equal, the strength would be the same as the "ideal." It is to be noted that in neither case was the proportion of cement increased.

It is quite true that for a 1:2:4 mix, with a given coarse aggregate, a given strength can result from the use of but one and only one grading (fineness modulus) of sand, but it would be only by the remotest chance, if ever, that the sand would happen to be the one showing the maximum tensile (or compressive) strength in a 1:3 mortar.

From this viewpoint the conclusion as to what the graph shows would have to be revised somewhat as follows:

1. That there is no ideal grading of sands which produces maximum strength in concrete, generally speaking, for, with each variation in coarse aggregate, there should be a corresponding change in the fine aggregate, either in its grading (fineness modulus) or in the proportion used.

2. That an exceptionally high tensile strength due to grading only will give the maximum strength only when properly combined with a given coarse aggregate. When used in an arbitrary mix it is just as apt to lower the strength as to raise it.

It is easily seen that to get a certain strength from a given coarse aggregate using an arbitrary mix one's choice of fine aggregate is extremely limited. One must either find exactly the right sand or miss the strength desired. How much more sensible and economical it is to design a mix out of the materials available, using the minimum amount of cement with the maximum value of fineness modulus for the conditions at hand. Instead of being confined to a single grading for a given strength, the variety of usable gradings is very large. The proportion between the fine and coarse aggregates should depend entirely upon the nature (grading) of the aggregates and the grading of the combined aggregate desired.

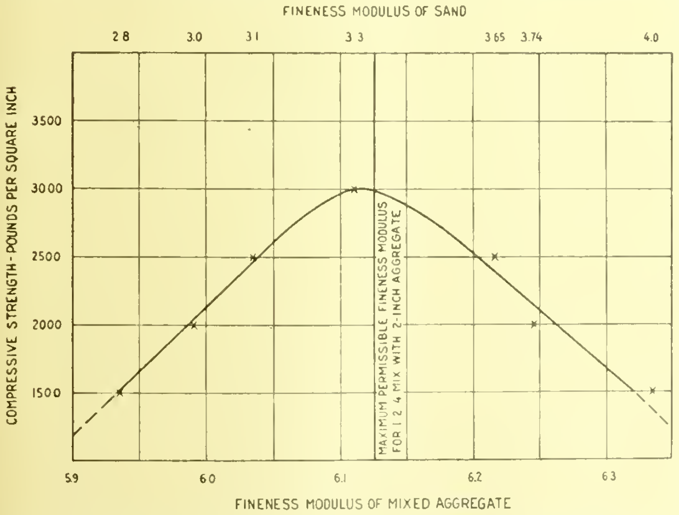

Fig. 2.—Relation Between Compressive Strength of Concrete and the Fineness Moduli of the Sands Represented by the Curves of Figure 1, and the Corresponding Fineness Moduli of the Mixed Aggregate Using Coarse Aggregate With a Fineness Modulus of 7.5 and a Maximum Size of 2 Inches

WHAT IS REALLY WRONG WITH THE 1:3 MORTAR TEST

The following argument is based largely upon data contained in Bulletin No. 1 of the Structural Materials Research Laboratory, Lewis Institute; and a bulletin covering nearly the same things issued by the Portland Cement Association. We believe that the water-cement ratio is the fundamental fact underlying all scientific methods of designing concrete mixes, and that the water-cement ratio is a function of the fineness modulus of the aggregate. However, if this theory were only partially correct there would still be considerable weight to the following consideration.

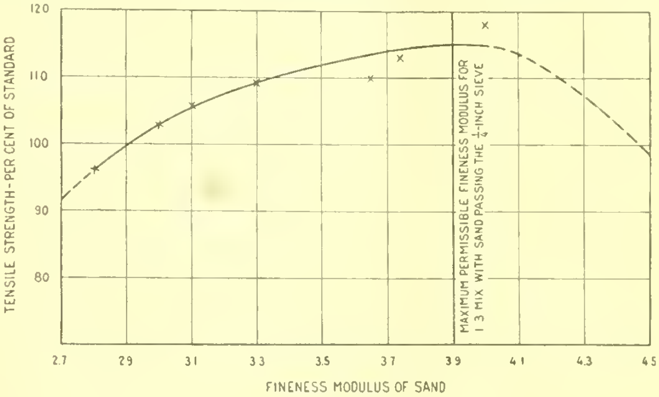

Fig. 3.—Relation Between Tensile Strength of 1:3 Mortar Specimen in Percentage of the Strength of Ottawa Sand Specimens and the Fineness Modulus of the Sands Represented by the Curves of Figure 1

Fine aggregate, or sand, is defined as that material passing the 4-mesh sieve. Our present specifications state that a 1:3 mortar made from a given sand shall develop at least 100 per cent of the strength of an identical mortar made with standard Ottawa sand. In other words, an aggregate having the maximum size of 4 must develop as much strength as Ottawa sand having a fineness modulus of about 2.95 and maximum size of 14. (See Bulletin No. 1 referred to above for a definition of maximum size of aggregate.)

The object of the 1:3 mortar test is to detect structural unsoundness. Occasionally it is said that it determines the suitability of the grading also, but, as will be shown later, a sand suitably graded for one arbitrary mix may be wholly unsuited for some other mix; and, when using the sand in a scientifically designed mix, a sand that will not pass in the 1:3 mortar test may be very well used if combined in the correct proportion—if structurally the sand is sound enough.

Let us see whether with our present method of testing we are able to detect structural unsoundness, or even to determine the suitability of a sand. Examination of hundreds of reports of sand tests shows that the average sand graded from zero to four must have a fineness modulus of about 3 as a minimum in order to develop as much strength as standard mortar. There are finer sands than this that will pass the test, but, as a rule, these sands do not fall into the class of zero-to-four grading. A sand having the same maximum size as Ottawa (14) and having a fineness modulus of 2.5 should, if structurally as sound as Ottawa excel the standard strength. This is due to the fact that 2.5 is the maximum permissible fineness modulus for a zero-to-fourteen aggregate and a 1:3 mix. Ottawa sand has a fineness modulus of about 2.95 and is therefore too coarse to develop its maximum strength. The sand with the 2.5 fineness modulus and maximum size of 14 would therefore excel it. (See table of maximum permissible values of fineness modulus in Bulletin No. 1.) However, the average sand graded from zero to four must have a fineness modulus of about 3 as a minimum in order to pass the test. A modulus smaller than this will raise the water-cement ratio to a point which will disqualify nearly all sands.

It is a significant fact that with a 1:3 mix and an aggregate graded from zero to four the maximum permissible value of fineness modulus is 3.90. This means that sands may range in grading from 3 to 3.90 without chance of failure due to grading. It is, of course, obvious that any sand with a modulus between 3 and 3.90 which fails to pass the test is structurally unsound. But what of the sand coarser than 3 that passes, and the sand finer than 3 that fails? An example or two will help to answer this.

Suppose that we have a sand with a fineness modulus of 3.60, graded from zero to four, which passes the test at 120 per cent. Is the sand structurally sound? We have always assumed that it is because it has developed more than standard strength, but how do we know what it would have done had the fineness modulus been reduced to 3? In other words, the grading of this sand permitted a lowering of the water-cement ratio enough, perhaps, to more than account for the 20 per cent over the standard strength. Suppose, for the sake of argument, that this sand were actually unsound, but that due to the lowered water-cement ratio it was able to pass the test. This is not at all improbable because the difference in the amount of water required for a fineness modulus of 3 and one of 3.60 is considerable. We now, having accepted the sand for use, combine it with a coarse aggregate in such proportions as to produce a certain fineness modulus in the combined aggregate (Abram's method), or we combine it according to some arbitrary mix. Let us consider the first alternative.

EXAMPLES SHOW DEFECTS OF MORTAR TEST

By Abram's method the sand is combined entirely on a basis of grading (fineness modulus), taking into consideration the fineness modulus of the fine aggregate, of the coarse aggregate, and of the combined aggregate to be produced. The aggregates are assumed to be sound structurally. Now that the combination is made we are no longer dealing with a sand graded from zero to four and having a fineness modulus of 3.60, but with an aggregate which has, say, a fineness modulus of 5.8 and a maximum size of 1½. The sand is now an integral part of the combined aggregate and has lost the advantage in grading that it had in the 1:3 mortar test. An inferior concrete is the result because the sand was unsound.

If we adopt the second alternative, and mix the sand in combination with a coarse aggregate according to some arbitrary mix such as 1:2:4 it is possible that not only will the actual unsoundness of the sand lower the strength, but also that *the very grading that caused the sand to pass the test may be a disadvantage in the arbitrary mix.* An example will make this clear. Given:

Coarse aggregate—	
Fineness modulus	7. 50
Maximum size	1½
Fine aggregate—	
Fineness modulus	3. 60
Maximum size	4
Mix	1:2:4

The above materials combined in a 1:2:4 mix produce a combined aggregate having a fineness modulus of 6.12. Now, the maximum permissible value of fineness modulus for this mix and maximum size is about 5.8. The above mix is therefore too coarse to produce maximum strength. The aggregate is undersanded. There should be either more of this sand added (change the mix to 1:2.6:3.4) or else the same proportion of a finer sand should be used. If the modulus of the sand were 2.4 instead of 3.6 the fineness modulus of 5.8 would be produced without changing the mix. All of this is based on the fact that maximum strength for a given mix is produced when the aggregate has the maximum permissible fineness modulus for that mix. If the value is lowered the water-cement ratio is raised; if the value is raised without increasing the maximum size of the aggregate there is not enough cement to fill the voids.

In the preceding paragraph we found that a sand having a fineness modulus of 2.4 would have produced a maximum strength with that mix and that coarse aggregate. Suppose that a sand graded from zero to four and having a fineness modulus of 2.4 were submitted for test. Let this be a clean, sound, sand. Experience has shown that in all probability this sand would fail in the standard test because of its fineness and consequent raising of the water-cement ratio. The sand would be rejected and in so doing we would be rejecting the only sand that could be combined with the coarse aggregate postulated above to produce the maximum strength for a 1:2:4 mix. This is an important consideration in view of the prevalence of arbitrary mixes. Furthermore, in rejecting this sand, we would be discarding material that might very well be used with any coarse aggregate in a properly designed mixed.

The fallacy in trying to determine the suitability of a sand by the present method is apparent. We have seen how an unsound sand could be accepted for use, and how a sound, useful, sand could be rejected. The gradings suitable in the test might be wholly unsuitable for the concrete, and vice versa. Surely there is a crying need for a test for sand in which the *quality* of the sand is the only variable, and by which the *degree* of soundness can be measured.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

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*136D. Highway Bonds. 20c.
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257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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Vol. 5, No. 19, D- 3. Relation between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPTEMBER 30, 1926

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927										BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION					
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES			
Alabama	$ 18,226,411.34	$ 8,725,985.09	1,298.3	$ 1,033,288.73	$ 492,916.35	75.7	$ 4,136,485.77	$ 1,908,310.79	133.1	$ 1,704,203.72	$ 852,101.34	138.8		$ 2,370,141.43	Alabama
Arizona	10,949,878.26	5,863,772.35	729.8	272,263.37	172,004.74	26.7	1,447,874.27	994,768.21	81.0					2,586,703.70	Arizona
Arkansas	18,364,544.50	7,666,698.36	1,323.0	527,162.26	258,399.12	38.8	4,338,696.69	2,052,855.61	279.9	1,250,921.81	621,849.50	94.2		1,006,001.62	Arkansas
California	27,142,596.90	13,003,592.30	1,058.0	2,030,143.39	1,015,180.72	81.6	10,385,227.30	5,079,749.26	248.7	654,219.57	334,197.44	16.0		2,639,095.28	alifornia
Colorado	13,906,904.64	7,127,288.18	745.0	703,326.39	382,146.82	25.5	6,366,219.12	2,579,873.46	236.0	328,001.35	183,954.34	27.9		2,052,549.20	Colorado
Connecticut	5,414,667.19	2,100,585.80	117.1	356,224.19	137,002.12	6.8	4,136,290.50	1,127,030.91	53.5	881,694.24	225,393.21	10.7		743,668.96	Connecticut
Delaware	4,918,052.29	1,781,665.60	124.3	672,626.56	275,874.75	17.5	769,795.60	334,016.25	17.9	243,288.10	71,705.55	8.3		10,785.85	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	982,922.96	489,063.29	31.9	8,943,871.42	4,154,236.83	247.8	658,042.07	331,803.95	10.1		1,285,488.61	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	1,551,142.10	761,763.40	91.3	12,161,961.91	5,914,844.66	672.5	100,093.62	50,046.78	12.8		41,060.41	Georgia
Idaho	11,061,198.14	6,882,112.70	724.7	329,995.93	170,379.07	13.3	2,940,022.84	1,774,123.93	166.3	899,794.46	523,839.66	67.4		209,171.64	Idaho
Illinois	44,116,611.86	20,619,996.74	1,377.7	2,012,307.84	980,960.66	68.1	7,992,570.40	3,869,179.54	283.7	1,326,334.89	634,979.47	46.2		3,727,082.59	Illinois
Indiana	16,949,426.87	8,172,125.19	534.3	2,165,077.11	1,022,048.82	65.2	17,281,911.36	8,140,398.89	501.4	105,513.96	52,756.98	0.4		817,025.12	Indiana
Iowa	29,062,375.40	11,926,302.10	2,114.8	1,478,825.10	677,219.97	85.6	12,240,818.19	6,631,714.11	713.9	2,320,724.77	969,866.23	78.3		280,460.59	Iowa
Kansas	32,826,601.64	12,590,489.25	1,160.6	522,940.32	235,898.35	48.9	12,903,599.34	5,060,211.47	703.2	2,777,042.82	969,942.08	134.9		607,869.85	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	307,209.39	106,386.83	6.3	7,814,740.29	3,658,046.70	372.7	436,195.46	217,597.72	21.2		738,695.60	Kentucky
Louisiana	13,830,592.68	6,144,739.99	1,055.9	100,810.58	47,898.26	10.0	4,139,744.21	2,009,768.27	197.3	1,212,069.75	517,750.40	38.8		552,261.08	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6	294,012.93	116,469.49	11.6	2,615,984.17	951,520.63	73.3	1,313,319.36	476,921.22	38.1		727,409.37	Maine
Maryland	10,924,943.10	5,112,991.22	423.3				1,350,356.90	641,523.57	72.2	426,154.66	170,540.72	16.1		1.49	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.5	58,723.21	22,710.05	0.2	6,208,753.36	1,397,642.65	73.3	773,461.02	198,908.17	11.9		1,832,204.51	Massachusetts
Michigan	25,997,240.78	11,827,062.30	963.0	390,946.77	188,717.04	13.6	13,317,248.82	6,086,174.43	378.7	423,878.96	161,472.69	4.7		2,088,948.54	Michigan
Minnesota	37,170,985.95	15,586,116.56	3,161.9	2,469,492.66	1,130,159.07	228.6	7,383,336.64	2,724,700.00	362.6	407,723.04	6,000.00	16.2		144,804.37	Minnesota
Mississippi	16,146,088.52	7,414,534.10	1,129.0	769,794.60	377,270.65	45.3	7,619,272.14	3,717,199.28	381.9	681,391.39	283,814.10	48.4		335,199.87	Mississippi
Missouri	28,989,166.92	13,736,014.85	1,543.2	4,214,395.17	1,841,217.99	123.8	16,904,839.84	6,712,627.78	462.0	671,100.91	291,025.70	13.4		205,549.68	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	384,690.63	258,568.03	48.9	2,405,431.31	1,900,630.92	166.5	861,904.17	476,823.50	102.0		4,455,396.66	Montana
Nebraska	11,533,401.62	5,474,202.62	1,768.3	593,394.96	291,575.09	65.7	13,823,916.11	6,814,242.74	1,448.9	518,097.23	258,819.11	69.2		1,796,395.54	Nebraska
Nevada	7,658,195.51	5,130,934.59	538.8	1,219,911.65	1,062,768.79	115.0	2,470,999.94	2,119,621.70	290.2	36,304.46	31,846.26	4.9		450,143.66	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6				1,401,209.12	640,606.43	41.8	81,083.86	35,258.94	1.1		116,176.56	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	853,531.14	202,054.20	13.3	8,170,401.15	2,721,490.84	44.1	1,291,908.66	298,965.00	19.9		146,567.75	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0				2,371,922.68	1,588,165.06	161.2	501,479.13	340,045.71	63.6		1,704,517.86	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	1,876,003.91	676,396.18	42.0	35,703,573.00	9,594,450.20	607.1	9,236,650.00	2,087,355.00	129.1		3,775,036.43	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9	3,621,683.44	1,412,891.77	73.3	4,970,040.28	2,262,344.60	132.7	972,106.70	456,189.10	40.4		408,442.59	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	1,012,765.16	469,539.29	102.6	6,880,714.72	3,641,076.33	932.5	1,058,913.12	538,327.89	148.9		67,855.71	North Dakota
Ohio	47,699,632.90	17,371,787.03	1,364.1	1,298,371.21	554,351.87	49.5	12,506,409.27	4,841,077.91	370.3	2,745,719.70	930,242.79	71.6		2,034,336.40	Ohio
Oklahoma	28,247,960.33	13,159,999.15	1,178.9	679,879.65	331,291.15	27.7	2,422,693.50	1,128,971.02	99.7	1,873,582.72	845,342.12	131.9		594,183.56	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	702,784.65	403,640.92	23.7	3,018,523.75	1,635,338.95	120.6	232,363.19	139,417.91	7.4		107,734.43	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1,188.8				28,932,350.11	8,216,400.41	578.7	1,476,142.34	454,809.16	28.0		1,106,839.39	Pennsylvania
Rhode Island	3,988,615.09	1,658,829.06	86.7	207,117.54	86,175.00	5.7	1,282,981.70	340,980.00	22.8	743,891.14	201,196.00	13.4		480,389.94	Rhode Island
South Carolina	16,020,639.90	6,766,322.93	1,481.9	731,575.09	262,587.41	44.2	5,923,078.58	2,673,278.57	205.1	309,121.16	56,921.68	13.4		43,413.51	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,181.2	677,690.35	289,877.22	127.1	4,026,484.17	2,041,920.59	676.2	79,488.88	39,744.44	14.1		191,420.78	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	757,678.17	353,743.87	21.3	8,490,357.75	3,770,336.51	243.6	390,741.12	185,871.09	8.0		694,055.51	Tennessee
Texas	69,183,673.48	27,440,264.72	4,920.2	2,092,131.01	939,622.50	165.2	17,143,916.84	7,563,979.60	764.6	3,203,769.21	1,494,511.91	87.4		3,178,062.27	Texas
Utah	8,253,178.03	5,098,440.68	546.4	72,006.59	53,284.87	9.1	1,883,615.88	1,417,258.71	170.4	853,136.36	656,272.37	49.7		593,522.37	Utah
Vermont	4,242,042.64	2,017,699.51	134.5				2,120,394.45	819,046.58	39.2	64,871.97	24,914.65	0.8		406,847.26	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.5	761,010.78	369,368.50	28.9	6,146,256.68	2,613,398.01	172.4	67,455.62	26,640.23	0.9		116,379.15	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	267,789.25	101,642.49	4.3	3,303,868.75	1,635,600.00	43.5	1,008,163.90	374,000.00	43.7		251,624.05	Washington
West Virginia	9,473,716.44	4,141,062.65	382.9	457,126.16	193,696.41	12.5	5,559,201.30	2,196,923.30	153.8	986,481.20	454,756.64	45.0		366,072.00	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1	101,729.42	50,664.71	9.3	8,467,314.02	4,034,936.12	379.1	591,446.04	270,722.50	24.4		2,689,785.94	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.5	465,830.26	297,267.00	60.8	2,739,880.13	1,741,877.34	206.2	78,837.13	50,612.00	19.1		435,630.61	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9					787,517.82	Hawaii
TOTALS	966,692,834.36	426,176,703.58	52,526.6	41,987,432.47	19,553,694.83	2,166.4	364,636,044.09	154,776,992.53	14,699.0	48,755,818.93	18,865,072.15	1,992.6		52,000,536.91	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 80,739,378.86 Federal aid $ 38,751,808.47 Miles 3,390.4

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A22.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 9 ▽ NOVEMBER, 1926

North Carolina's Recent Economic Progress is Largely the Result of the Improvement of its Roads

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1926

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 7, NO. 9 NOVEMBER, 1926 H. S. FAIRBANK, Editor

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FIVE YEARS OF STATE ROAD BUILDING AND ITS RESULTS[1]

By **FRANK PAGE**, Chairman, State Highway Commission of North Carolina

The building of our State system of highways in North Carolina during the last five years has linked together our great trunk-line railroads and our waterways into an effective transportation machine.

On the coast our ports receive imports from all the world distributed to our Carolina markets by rail and motor. A coastwise shipping takes potatoes, melons, and truck produce to Baltimore, Washington, and New York.

Our trunk-line railroads take our fabricated tobacco, cotton, and furniture from the factories of Gastonia, Durham, Winston-Salem, and High Point and deliver them to all parts of the United States and other countries.

Our 6,500 miles of State highway system—4,500 miles improved plus the secondary feeder roads—are the means by which much of the raw materials get to the factories and thence to the rails. Conversely they are the means of distribution of our imports and food products to our people.

In this period of five years from 1921 to 1926, North Carolina has probably made greater progress agriculturally, industrially, and socially than any other State in the Union in a similar period of time.

One of the most important factors in this metamorphosis has been our highway improvement program, involving the building of $125,000,000 worth of highways since 1919—most of it since 1921 when we issued $50,000,000 in bonds and later $35,000,000 more to build a State system of highways.

What has been the result? Has it helped or hurt industry, agriculture, the taxpayer, the 4,000 miles of trunk-line railroads and 1,000 miles of short lines in the State?

A few facts will help us to understand what has happened since 1920 and 1921.

We have increased the number of our farms by 13,000 during a period when the number of farms for the country as a whole fell off.

We have built consolidated rural schools valued at $35,000,000. To these each school day are brought 100,000 pupils in 2,000 school busses operating 40,000 miles—largely on the State system.

We have developed 40 cooperative farm marketing associations, many of them stimulated by our State Department of Agriculture, and engaged in shipping carload after carload of poultry, eggs, hogs, fruits, and vegetables that we formerly never grew for outside sale.

Roadside markets and city curb markets have stimulated the growing of truck produce on our farms and are the outlet for the farm surplus. The farm women are the merchandisers of this surplus and with the first available cash money most of them have ever had are putting modern conveniences into their homes, dressing themselves and their children better, painting their houses, and beautifying their yards.

One of the Sand-Asphalt Roads Recently Constructed in the Coastal Section of the State

We have recovered our lost provinces—those sections of the State to the far east and west formerly foreign to the State so far as transportation connections of any kind were concerned. To reach some of these points in our own State involved rail travel through one and sometimes two adjoining States. Our highways were out of the question for travel to these places. To-day the roads have gone through and we have put every part of the State within reach of every other part—almost from sunrise to sunset over the State highway system.

The State of North Carolina has taken the lead in spindle hours operated by cotton mills.

The true value of our property has multiplied eight times since 1900 while the entire country was increasing the true value of property by four times.

The question may arise as to how far the highways can be credited with bringing this about. Let us analyze the situation further.

HOW WAS HIGHWAY IMPROVEMENT ACCOMPLISHED?

We have built our State system and are maintaining it entirely at the expense of the road user.

There is no State property tax for highways. State highway moneys come from three sources—bonds, motor registration fees, and gas taxes. The bonds are issued serially and are retired through a sinking fund maintained entirely by the motor taxes.

In 1921 the State legislature established a State highway system, 6,500 miles in length and tapping every county seat and every town and city of importance. The State is now maintaining 6,200 miles of road—all but some 300 miles of mountain trails. The roads first improved were naturally those between the cities because communication between them was necessary before tying in the more remote parts of the State.

North Carolina is predominantly agricultural, with manufacturing centered in the Piedmont section around Durham, Winston-Salem, Greensboro, Charlotte, Gastonia, High Point, and Raleigh. Our highway system makes our industry and agriculture interdependent—each supplying the other. The lumber for our furniture factories, cotton for the cotton mills

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[1] A statement made by the writer in hearings before the Interstate Commerce Commission on interstate operation of motor trucks and busses, I. C. C. Docket 18,300.

Before the Roads Were Improved the Mountainous Section of the Western Part of the State was a Lost Province. To Reach It by Highway was Impossible. To Reach It by Rail One Traveled Through Two Adjoining States

and tobacco for the great factories at Durham and Winston-Salem are produced within the State and move both by rail and highway at the beginning and end of fabrication.

INFLUENCE OF HIGHWAY IMPROVEMENT ON AGRICULTURE

In 1925 there were 283,491 farms in North Carolina with nearly 7,000,000 acres under cultivation. This cultivated area represents about 27 per cent of the area in farms and about 23.5 per cent of the taxed area of the State.

The three leading money crops—cotton, corn, and tobacco—occupied 72 per cent of the cultivated area of the State, and with grain and hay constituted 92 per cent of the cultivated area.

Diversification, however, is becoming more popular and improved highways now make available the markets for the products of diversification. Some 40 cooperative farm marketing associations, mostly organized since 1920, with the excellent aid and advice of the State department of agriculture are the medium of much produce selling. With the exception of the farmers' federation at Asheville it is doubtful if any of them operate their own trucks. Roadside markets are fast becoming a factor in disposing of farm surplus products.

William A. Graham, State Commissioner of Agriculture, stated:

> The good roads in this part of the State have made it possible to supply the manufacturing cities from farms located as many as 40 miles away or farther. Prior to the advent of good roads the farmer who lived as much as 10 miles from town rarely took produce to market unless his roads were in what he would call prime condition, and then it took him an entire day to make the trip.
>
> The town was then forced to get its supplies from sources outside the State, as it could hardly draw on more than 75 square miles of territory for local production. With the coming of good roads the market gardens of the cities have grown from the former area of 50 to 75 square miles to 1,000 to 1,200 square miles or more. Here we find diversified farming—cotton, corn, tobacco, potatoes, wheat, oats, and vegetables—all growing on the same farm the same year.

Mr. Graham's statement forcefully illustrates the very large intrastate movement of produce from farm to city and of manufactured commodities from city to farm in return, almost entirely by motor truck.

The total value of North Carolina crops last year was $320,000,000.

In 1911 this Farmer Required Four Horses to Haul 520 Feet of Lumber 5 Miles. The Trip Took Half a Day and the Farmer Received for His Lumber $2 a Hundred

But Now the Roads Have Gone Through and Have Put Every Part of the State Within Reach of Every Other Part—Within a Day's Journey

During the year 1925 North Carolina truckers shipped to points out of the State 15,555 carloads and 490,000 express packages of truck, which brought nearly $12,000,000 direct to the producers.

The highways have been the medium for no small amount of new agricultural tonnage reaching the rails. This is particularly true of poultry and hog shipments. In 1924 we shipped out of North Carolina 464,285 pounds of poultry, in 1925 some 750,000 pounds, and in 1926 (year ended June 30 in each case) the total reached 1,900,000 pounds, 250 per cent more than the 1924 figure.

As to hogs, in 1924 there were shipped 23 carloads, in 1925 this increased to 58, and in 1926 to 80 carloads, or more than three times the 1924 figures. Just the other days one of our State college men told of seeing 32 farmers receive a check for $11,000 covering the shipment of eight cars of hogs they had brought in to three rail-shipping points by motor truck.

Shipment of eggs in 1924 totaled 6,681 cases and increased to more than 8,000 cases in 1926.

Dairying.—The State has lately turned strongly to the possibilities of dairying. It goes without question that highway improvement has been quite largely responsible for getting large quantities of milk into our southern cities. The larger ones show an increase of 250 per cent in milk consumption.

Nine of the principal cities of the State now pay farmers of surrounding sections $4,178,520 annually for milk. Incidentally, this supply has been developed and handled almost entirely over the highway so that the railroad has not entered as a competitive transportation factor.

The highways have assisted in the establishment of milk plants. John Arey, of the State college, reports the establishment of seven milk plants in the State since 1920. There were but five at that time. There are now 15 creameries making butter, 7 of them also established since 1920, with others now under construction.

Some 7,500 farmers sell cream to these establishments with an annual butter production of 2,000,000 pounds.

Fruits and Vegetables.—The United States Bureau of Agricultural Economics reports car-lot shipments of fruits and vegetables out of North Carolina for the five-year period 1921 to 1925, inclusive, as follows (some of the smaller lots are not included, which accounts for the difference between this total and one previously given for 1925):

Car-lot shipments of fruits and vegetables from North Carolina

Crop	Year				
	1921	1922	1923	1924	1925
Apples	77	384	220	433	344
String beans	128	219	261	559	459
Cabbage	251	213	364	263	371
Canteloupes	894	700	620	401	655
Cucumbers	641	687	1,175	1,639	1,562
Lettuce	445	622	718	714	537
Mixed vegetables	66	326	758	1,093	853
Peaches	594	1,452	215	1,657	2,024
Spinach	2	5	25	21	11
Strawberries	503	1,101	1,068	2,046	1,634
Sweet potatoes	982	721	654	720	1,189
Tomatoes	1			8	
Turnips and rutabagas	6	9	5	2	3
Watermelons	1,657	993	1,542	664	991
Potatoes	3,071	4,202	3,475	6,566	4,052
Total	9,318	11,634	11,700	16,786	14,685

There is of course evident the effect of crop and market fluctuation, particularly in potatoes in 1924, but the sum total has been steadily increasing. Much of this tonnage moved to rail points over the highways. Some, particularly the potatoes, moved by water to rail shipping points. Each year, however, a greater percentage of certain crops comes by motor truck from longer distances to rail-shipping points.

Peaches. The sandhills of North Carolina produced an enormous crop of peaches this year. Unfortunately the Georgia crop was a little late so that our peaches got into northern markets about the time theirs did, depressing the price to all producers.

Here the highways were of considerable value, not so much in the car-loading of peaches as in the distribution of the surplus throughout the State. The truck was a safety valve, which handled the equivalent of 100 carloads or more of the grades which could not be shipped by rail. These went by truck direct to home markets. Another 10 to 15 carloads were shipped by rail to other cities in the State and from there marketed by truck.

At Present Cotton Can be Moved at Any Time. The Condition of the Road Plays Little Part Except to Cheapen the Cost of Hauling the Raw Cotton to the Gin

Five years ago the peach growers would have taken their loss, for the highways were not there to help them dispose of this grade of peaches. And too, the rails did not lose the business, because most of these peaches were too ripe to ship by rail.

ROADSIDE MARKETS PROVIDE NEW OUTLET FOR DIVERSIFIED PRODUCTION

At Hot Springs, N. C., there is a farm of 4 acres that two years ago, prior to putting through an improved road, produced little more than enough to support the farmer and his family who lived on it. This year they have not only lived well on it, but have cleared $2,200 in cash by marketing at their front gate the fruit and vegetables they produced.

Another farmer, operating a farm of 200 acres at Awanonoa has been clearing $25 a day for the summer and fall from the surplus produced on his farm. Here the market has no attendant but is conducted on the honor system and the owner claims never to have lost a cent. Prices are indicated and the buyer makes his own change.

Another farmer in Alamance County daily brings his cream 56 miles into Raleigh and delivers it to a creamery.

In Anson County a farm woman is canning goods at the farm. She travels 12 miles with a horse and wagon over the improved highway and sells her canned goods at the curb market in Wadesboro. For 10 years prior to the bettering of this road she had not been in Wadesboro more than twice. Now with the first cash she has seen for years, she is buying clothes for herself and children and conveniences for the house.

We have now 29 curb markets in different cities at which the farmers last year sold nearly a quarter of a million dollars worth of surplus produce from the farm. Five years ago there were only two or three of these markets, and those only in the larger cities.

The roads around Rocky Mount, N. C., were improved a little more than three years ago. A curb market was established 31 months ago and to date has resulted in the sale of $32,500 worth of produce.

Another phase of this curb market development is the growth of community booths within them. In one of these booths, one farm woman sells $10 worth of cakes a week, another has cleared $150 this year on canned soup mixtures, while still another has made $520 from her garden and chickens.

Not the least of the advantages of these markets has been the social contact between the city and farm. Each has learned what the other is doing. The farm women use the receipts from their produce sales to buy conveniences for the farm home—clothing, books—everything they have wanted but could not get before for lack of funds. Incidentally, all these purchases make increased business for the railroads, waterways, and highways.

HIGHWAY SERVICE AND THE COTTON AND TOBACCO INDUSTRIES

Cotton.—Just now we are faced with an over-production of cotton with attendent ill effects. The doctrine of diversification is being advocated. Our cotton fields are small and the soil is adaptable to other products so that we have all conditions favorable to production. Marketing facilities exist in the cooperative associations holding out the buying power of distant markets; near-by cities have their curb markets and the highways are at hand over which both can be reached.

At present cotton can be moved at any time. The condition of the road plays little part except to cheapen the cost of hauling the raw cotton to the gin.

Tobacco.—The chief influence of the improved highway on the tobacco grower has been to change his marketing processes. Prior to this development there were usually one or two small tobacco warehouses in each marketing town. The tobacco grower came to these with his crop, taking all of one day with his mule and wagon to get there. The next day was spent in arranging the tobacco in baskets, grading and sorting it on the warehouse floor and closing sales with the buyers who visited these towns. The warehousemen cleared all sales and were therefore the commission merchants.

The grower now loads his truck at sunrise, travels up to 40 or 50 miles, puts his product on the warehouse floor, sells it and is back home again the same day. The process of marketing is shortened and cheapened; fewer buyers are required; the grower is always sure of a market at the larger point at any time he chooses to go; and he gets a better price for his product.

The industries of the State use rail transportation predominantly. They do, however, make frequent and necessary use of the highways.

One instance shows the service the highways rendered one cotton mill company, where the rail haul was circuitous and a delivery point a half mile from the railroad was involved.

In May this year a manufacturing company at Concord, N. C., bought 100 looms from a firm at Newton, N. C. The distance is 65 miles by highway and about 72 by rail.

The manufacturing company contracted with a trucker for $650 to take the looms from Newton to Concord, handling five every 12 hours and working night and day.

To ship by rail it would have been necessary to ship the looms single deck for lack of skilled labor to store them double deck. The freight charges between the two points on this basis were $515, plus $300 for labor and trucking from the station at the other end of the haul to the plant half a mile away, plus certain unknown charges for blocking the looms in the cars.

The Number of Farms has Increased by 13,000 During a Period when the Number Fell Off in the Country as a Whole

The two State highways and one county highway involved in the movement by truck were thus worth $165 to the manufacturing company in this one instance alone.

Most of the raw materials come to the cotton mills by rail and most of their finished products go out by rail. The highway, however, permits them to draw on a wider supply of labor, principally from the farm.

The truck line past the door is a guarantee against delays in productions and shipment. A telephone call to a near-by city, perhaps 40 to 50 miles away, for 100 feet of leather belting or a few feet of pipe with orders to deliver to the truck line and the needed supplies are in the factory a few hours later.

The other day a mill received an order for immediate delivery of goods to be shipped in boxes of unusual size. None was on hand. Time did not permit of ordering for rail shipment. The order for the box shooks was placed by telephone; the shooks were picked up by motor express that afternoon 40 miles away and delivered to the factory the next morning. By that afternoon the order received 36 hours before was on the railroad loading platform.

One other service is given the mills by the highway and motor transport. This additional transportation is a guarantee against mill shutdowns or delays in filling orders due to uncertain or undependable rail service, car shortage, etc. Cotton warehouses maintain their own trucks and when a mill needs raw cotton urgently it is sent by motor truck. It costs more to send it that way, but this cost is offset by the mill's ability to keep running and meet consumer demands on it.

Cottonseed oil.—The cotton gins are usually located near the fields and several of them will supply one oil mill with seeds. Formerly the seeds were hauled to a rail siding or into town to the rail freight station when the gin itself was not on a siding. Now it is not infrequent to haul the seeds by truck direct from the gin to the cottonseed oil mill, thereby avoiding extra handling delays, etc.

Furniture. The center of the furniture industry is at High Point and is served almost entirely by rail, both as to raw materials and finished product. One exception is worthy of note. At Asheboro and Thomasville in particular (near-by towns) many split,cane-bottom chairs are made. The cane bottoms are sent out as piece work by truck to farms and residences in the neighboring sections. As the work is completed the bottoms are gathered by truck, carried back into town for assembling and rail shipment from there to market.

INFLUENCE OF HIGHWAY IMPROVEMENT ON CITIES

The smaller cities in particular have felt the impulse to growth called forth by the improving of the highways connecting them with larger and more progressive cities. Now nearly all of them have new hotels, new business houses, municipal office buildings, paved streets, and better homes. Some of the larger cities show this growth more specifically.

Raleigh is now the trading center for 172,000 persons, 31,000 of these within the corporate limits and the balance within an area of 25 square miles. The city's population has increased from 24,418 in 1920 to 31,000 as of July 1, 1926. Building permits have increased from $862,322 in 1920 to $3,502,011 in 1925. There has been $17,000,000 worth of building in the last five years with the total assessed valuation for the city in 1926 given as $45,000,000.

The relation of highway to rail during this period is shown by a statement from Plate Collins, secretary of the Kinston Chamber of Commerce, October 23:

> Volume of shipments by rail has increased 20 per cent since 1923 in spite of the fact that a big part of short-haul freight formerly handled by the railroad now goes by trucks.

The Use of Busses has Caused the Suspension of Some Local Train Service but has Not Materially Affected the Number of Through Passengers Carried by the Rail Lines

The Goldsboro Chamber of Commerce estimates "increase in business 25 per cent caused more and better homes both in city and country, increase in both mercantile and factory buildings."

From William T. Ritter, secretary of the Winston-Salem Chamber of Commerce, comes the following:

> Our retailers report a 65 per cent greater purchasing power per capita in our trade territory. The property valuation in our county has increased over $30,000,000 since 1921, and some $2,000,000 worth of homes are being erected by our citizens on areas located on the main highways serving this city. The outputs of our industries as well as agricultural areas have shown tremendous increases since 1921, and there is no gainsaying but that the State's highway program is responsible in no small measure for this increased prosperity.

John D. Topping reports from Asheville, one of the resort centers of the State, for the past eight months of 1926 as compared with the same period for 1925:

> The dollar volume of business shows 45 per cent increase over 1925—204 per cent increase over 1919. The following figures show increases in 1926 over 1925: Bank deposits, 26 per cent; post office, 22 per cent; building permits, 42 per cent. Seven hundred residences inside the city and an equal number outside being built. Number of visitors increased 25 per cent.

The secretary of the Greensboro Chamber of Commerce, says of Greensboro that it is to-day, "the natural traveling center for 1,500 salesmen and agents because of accessibility over good roads. This and other factors have created a demand for homes and other facilities costing more than $25,000,000 as represented by the volume of building permits in the last six years. Good roads have widened our retail-trade territory to an irregular area extending from 15 to 50 miles and has put more than half a million people within one hour of the city."

All this increased consumer demand has, of course, been beneficial to rails, manufacturers and wholesalers and retailers alike. We have, however, developed new methods of distribution, particularly in the short haul.

In the eastern part of the State a distributing company operates an extensive motor-truck service in putting its staple and perishable groceries in the retail markets.

At Charlotte, N. C., a chain-stores company distributes daily from its warehouses to its retail stores in North Carolina and South Carolina. It is said that the equivalent of 6 or 7 carloads of groceries is delivered in this manner under contract with a motor-truck operating concern.

MOTOR VEHICLE REGISTRATION

The registration of passenger automobiles was 127,405 in 1920 and is to-day 338,192, an increase of 265 per cent over 1920.

The State has registered now 33,161 trucks against 13,455 in 1920, an increase over that year of 247 per cent.

On July 1, this year, these vehicles had paid registration fees for the last fiscal year amounting to $8,630,754 and $6,898,548; in gasoline taxes, a total of $15,529,302 for the fiscal year ended June 30, 1926.

The highway system of the State was built primarily to meet the demand of the passenger vehicles for a better roadbed over which to operate. With the improvement of the roads, however, there has developed an economic dollars-and-cents return in commercial motor transportation. To the individual passenger-car operator there is a return of lower operating expenses, shorter time involved in trips, etc.

Potatoes and sweet potatoes are handled by boat and highway to the rail connection and thence to market by the rail line from the eastern part of the State.

Imports totaling nearly $14,000,000 came through the port of Wilmington last year, and were distributed to Carolina markets by rail and motor.

From 4 to 5 tons of fish are hauled daily by motor truck from Morehead City and other ports to points as far inland as 200 miles. Here the movement was formerly by express so that the highway has replaced the rail in distribution of fish from this point.

At New Bern, a packet company brings in sugar which requires the services of several trucks daily to distribute throughout the State.

RELATION OF HIGHWAYS TO RAILROADS

The year-round resorts of North Carolina have attracted a tremendous number of tourist visitors from all parts of the country. Pinehurst has almost an entirely rail movement of visitors. Golf, tennis, horseback riding, motoring, and particularly our mountain air are just as much salable commodities as our cotton and tobacco.

Golf, Tennis, Horseback Riding, Motoring, and Particularly Our Mountain Air are Just as Much Salable Commodities as Cotton and Tobacco

Hotels have sprung up rapidly in all our cities and resort centers to care for these visitors and the farmers in the surrounding country are turning to truck produce to feed them.

Some local train service has been reduced, yet in a survey of passengers carried by bus over a seven-day period, parallel to a rail line with excellent through service over a distance of 90 miles, the corporation commission found that the busses carried only 234 through passengers out of a total 6,115 handled between terminals in that period.

Many of our rail connections have been circuitous and the automobile and bus were substituted immediately, the highways permitted. For example, from Charlotte to Asheville (two of the largest cities in the State) is a distance of 120 miles by an improved road completed in 1923. To reach Asheville by railroad from Charlotte one must go through the States of Georgia and North Carolina via Spartanburg, S. C., a distance of 240 miles.

Williamston and Windsor are two towns of 1,800 and 1,200 population, respectively. They are 12 miles apart and separated by the Roanoke River. Prior to the completion of a bridge and improvement of the road between them in 1922, it required 24 hours to go from one town to the other. Now, the trip is usually made in as many minutes.

Unquestionably the passenger car and improved highways have greatly stimulated the traveling habit. Many people who formerly traveled by local train now have their own cars. Many who have tasted travel in their own cars try the longer distances by rail. Unquestionably most of the rail loss of passenger business has been in local business which has gone to the private automobile. We may well give attention to the question of substituting the bus for the local passenger train. In many cases this would relieve such lines of undue burdens in providing service for the small percentage of such business remaining.

Two other cities, Moorehead City and Beaufort are just 2½ miles apart—separated by an arm of Bogue Sound, yet the shortest highway connection is 45 miles. A rail line now connects the two, but the upkeep of the trestles which must be replaced very frequently because of destruction by the sea-water pests is understood to cost more than the revenue derived from the operation. A highway bridge three-fifths of a mile long, with highway connections is now being extended from Moorehead City to Beaufort. Undoubtedly all freight and passengers could here more economically be transferred to the highway.

Comparative figures for 1922 and 1925 are available from the State corporation commission records on some phases of the rail freight business handled within the State.

Comparative figures for three important trunk lines—the Atlantic Coast Line, Southern, and Norfolk & Western—for the years 1922 and 1925 show the tonnage originated on those lines within the State. Unfortunately the information is not available in a comparative way for years prior to the highway improvement begun in 1921. If it were, it is certain the increase in tonnage originated would be even more striking.

Tonnage originated within the State

Carload of—	1922	1925
	Tons	*Tons*
Products of agriculture	543, 680	603, 388
Animals and products	32, 035	46, 640
Products of mines	501, 411	698, 788
Products of forests	1, 683, 072	1, 750, 710
Manufactures and miscellaneous	1, 251, 534	1, 476, 724
Total carload	4, 011, 732	4, 576, 250
Less than carload	686, 145	683, 406
Grand total	4, 697, 877	5, 259, 656

As will be noted the 1925 carload figures show an increase of 564,518 tons over 1922, or 14 per cent increase in carload tonnage originated in the State.

The less-than-carload situation showed a loss of less-than-carload tonnage originated by the Atlantic Coast Line, but an increase for both the Southern and Norfolk & Western, the total not quite offset, however, by the loss sustained by the Atlantic Coast Line.

Beginning with 1921 our program of highway construction involved extensive rail hauling of cement, sand, and gravel. Also a tremendous tonnage of automobiles has been handled by these rail lines not alone in North Carolina but in the entire South.

Here again it is unfortunate that the figures can not be obtained for years prior to 1921, but as they stand they are illuminating as to the increased rail tonnage from these sources. The same three railroads are used; figures are given in tons of 2,000 pounds and do not include cement tonnage.

	Tonnage originated within the State		Received from connecting carriers	
	1922	1925	1922	1925
	Tons	*Tons*	*Tons*	*Tons*
Stone, gravel, and sand	451, 208	667, 140	1, 129, 147	1, 466, 826
Automobiles and auto trucks	1, 065	16, 992	42, 307	100, 736

The tonnage of stone, sand, and gravel originated within the State showed an increase in 1925 over 1922 of 215,932 tons or nearly 48 per cent whereas the tonnage received from connecting carriers increased nearly 30 per cent.

Tonnage of motor vehicles originated in the State is rather insignificant because so few vehicles are made there. The tonnage received from connecting carriers, however, increased 138 per cent. In both instances, although the percentage of increase is important the volume of this kind of business is the more significant item.

Movement of all kinds of freight within the State, both intrastate and interstate, is given below. The figures for only three roads were comparable for the years 1922 and 1925. Here again it is not possible to determine the percentage moving from outside points into North Carolina and from points in North Carolina to interstate destinations. The figures are in tons and are given for the Atlantic Coast Line, the Southern, and the Carolina, Clinchfield & Ohio.

	1922	1925
	Tons	*Tons*
Carload freight	20, 657, 105	24, 861, 828
Less-than-carload freight	1, 635, 168	1, 763, 373
	22, 292, 273	26, 625, 201

Many People Who Formerly Traveled by Local Train now have Their Own Cars. Unquestionably the Improved Roads have Stimulated the Traveling Habit

They cover all freight handled by these lines in the State of North Carolina, whether originating on these lines, part of interstate shipments from and to North Carolina points, or simply a trans-State movement.

ELECTRIC AND SHORT LINE RAILROADS

Just as the State reached a point of progress which attracted capital to project electric interurban lines the automobile came into prominence followed by the demand for road improvement. As a result these projects did not go beyond paper or in some instances the surveying. As a result we are to-day not faced with the problems in this respect that many States are.

Many of the branch steam lines and short-line steam railroads have served their purpose. People seem to prefer travel on the highway in their own cars and using their own trucks. It is time we recognized this and educated public opinion to the economic waste in forcing these lines to continue operations at a loss when the same public after voting to force continued operations, refuses to patronize the roads. The public must choose which type of transportation it wants and if the choice favors the highway there must be a closer coordination of rail and highway service.

(continued on p. 189.)

AN UNUSUAL CONSTRUCTION RECORD

STOP-WATCH RECORDS ON MISSOURI CONCRETE PAVING CONTRACT SHOW EFFICIENCY ABOVE 90 PER CENT

Reported by **C. F. ROGERS**, United States Bureau of Public Roads

During the 1925 construction season the Ross Construction Co., of Kansas City, Mo., Henry Kleifeld superintendent, completed a concrete paving job in Lafayette County, Mo., on which the standard of efficiency was so high that the results obtained deserve the acknowledgment of the widest possible publicity. One 5-bag mixer was used on this work. The batch was a standard 5-bag batch of the standard proportion of 1 : 2 : 3½. The pavement section was the standard Missouri Maricopa section with dimensions of 9 by 6 by 9 inches.

During the full construction season a little over 24 miles of pavement were laid, not quite all of it on this project. At no time was more than one mixer used, although the company had an extra mixer available which was used as stand-by equipment. The records obtained by the Bureau of Public Roads do not cover the full year's work, which was continued for something more than a month beyond the period for which the record was kept. However, the balance of the record would add little or nothing in the way of information to what is here given.

TABLE 1.—*Daily record of concrete road construction on Missouri Federal-aid project 228*

Section	Date (1925)	Hours of work	Length of pavement laid	Average length of pavement laid per hour	Weather and remarks
			Feet	*Feet*	
F	Apr. 22	5	381		Fair and warm.
	23	10	877		Cool and cloudy.
	24–25	Idle.			Rain.
	26	10	883		Fair and cool.
	27–28	Idle.			Rain.
	29	4	315		Stop, 40-mile wind.
	30	10	909		Fair and cool.
Total for month		39	3,365	86.3	
	May 1	10	1,006		Fair and warm.
	2	10	1,005		Do.
	3	10	978		Do.
	4	10	1,003		Cool and cloudy.
	5	10½	1,034		Fair and cool.
	6	10	965		Cool and cloudy.
	7	5	491		Rain in morning.
	8	5½	494		Rain in afternoon.
	9–10	Idle.			Rain.
	11	10	991		Fair and cool.
	12				Move to section E.
E	13–14	Idle.			Rain.
	15	5	392		Rain in afternoon.
	16–17	Idle.			Rain.
	18	5	434		Wet subgrade, morning.
	19	10	997		Fair and cool.
	20	11	1,104		Fair and warm.
	21	10½	1,027		Do.
	22	10	1,000		Do.
E and F	23	11	1,102		Cool and cloudy.
F	24	Idle.			Stockpile low.
	25	10½	1,072		Fair and cool.
	26	10	1,015		Fair and warm.
	27	10	968		Cool and cloudy.
	28	8½	835		Cloudy, 4.30 rain.
	29	1	46		Connecting slab.
	30 and 31				Move to section D.
Total for month		183½	17,959	97.8	
D	June 1	1	56		Cloudy, warm, rain
	2–3	Idle.			Wet subgrade.
	4	5	345		Fair and warm.
	5	10	840		Do.
	6	10	884		Fair and hot.
	7	10	894		Do.
	8	10	914		Do.
	9	10	883		Bridge approach.
	10	10	696		Do.
	11	10½	1,066		Fair and warm.
	12	5½	542		Rain in afternoon.
D	June 13	Idle.			Wet subgrade.
	14	10	989		Fair and warm.
	15	10	952		Fair and hot.
	16–19	Idle.			Rain and wet subgrade
	20	10	964		Fair and hot.
	21	10½	1,038		Do.
	22	4	360		Rain in afternoon.
	23	Idle.			Wet subgrade.
	24	7	654		Rain.
	25	10	1,024		Fair and cool.
	26	10½	1,040		Fair and warm.
	27	10	1,031		Do.
	28	8½	834		Do.
	29	Idle.			Rain.
	30	10½	1,027		Fair and warm.
Total for month		183	17,033	93.0	
	July 1	10	887		Fair and hot.
	2	10	953		Do.
	3	10½	1,003		Do.
	4–5	Idle.			Rain and wet subgrade.
	6	10½	1,045		Fair and warm.
D and E	7	10½	1,060		Fair and hot.
	8	10	1,012		Do.
	9	9	863		Do.
	10	10	955		Do.
	11	10	782		Bridge approach.
	12	9	846		Rain, 5 p. m.
	13	2	155		Rain, 9.30 a. m.
	14	Idle.			Wet subgrade.
	15	10	856		Heavy subgrade.
	16	10	913		Fair and warm.
	17	10	900		Do.
	18	10	979		Do.
	19	Idle.			Low water supply.
	20	1½	94		Rain, 10 a. m.
	21	5	469		Wet subgrade, morning.
	22	10½	1,060		Fair and warm.
	23	10	1,015		Do.
	24	10½	1,050		Do.
	25	9	798		Cloudy and cool.
	26	Idle.			Rain.
	27	10	932		Fair and warm.
	28	10½	1,048		Do.
E and F	29	4	335		Move to section F.
F	30	9	906		Rain.
	31	7	635		Wet subgrade, morning.
Total for month		228½	21,551	95.6	
	Aug. 1	10	972		Fair and warm.
	2	10	990		Do.
	3	5	503		Cement low.
	4	10½	1,057		Fair and warm.
	5	10½	1,088		Do.
	6	10	1,000		Do.
	7	5½	499		Rain in morning.
	8	11¼	1,151		Fair and warm.
	9	6½	614		Move to section B.
	10–13				Do.
B	14	7	400		Fair and warm.
	15	11¾	1,129		Do.
	16	10½	907		Do.
	17	Idle.			Rain.
	18	10½	980		Fair and warm.
	19	10½	989		Do.
	20–21	Idle.			Rain and wet subgrade.
B and C	22	10½	990		Fair and warm.
C	23	10½	985		Do.
	24	10½	1,071		Do.
	25	10½	1,056		Do.
	26	10½	1,023		Do.
	27	11½	1,089		Do.
	28	10¾	1,068		Do.
	29	10½	1,060		Do.
	30	9½	906		Move loader.
	31	10½	1,006		Fair and warm.
Total for month		234¾	22,533	95.9	
	Sept. 1	10½	1,002		Do.
	2	10½	998		Do.
	3	10½	988		Do.
	4	10	934		Do.
	5	10½	1,064		Do.
	6–7	Idle.			Rain and wet subgrade
	8	9	863		Fair and warm.
	9	10¾	1,073		Do.

Table 1.—*Daily record of concrete road construction on Missouri Federal-aid project 228*—Continued

Section	Date (1925)	Hours of work	Length of pavement laid	Average length of pavement laid per hour	Weather and remarks
			Feet	*Feet*	
C	Sept 10	7	651		Cloudy, rain.
	11-13	Idle.			Rain.
	14	5	443		Rain, afternoon.
	15	10½	1,038		Fair and warm.
	16	11½	1,110		Do.
	17	10	978		Do.
C and D	18	10	834		Do.
Total for month		125¾	11,976	95.2	
Total		994½	94,417		
Average		94.94 feet per hour for 994½ hours.			

Table 1 shows the number of hours of work each day, the number of lineal feet of pavement laid, and the reasons for such delays as were encountered. The table also shows the total number of days of work, the total number of feet laid, and the average number of feet laid per hour for each month and for the whole period. The record of 94.94 feet laid per hour for 994 hours of operation with 42.2 batches per hour for every hour worked is the highest production that bureau representatives have ever found.

Table 2 is a stop-watch analysis of a perfect hour's operation—the only one ever encountered in a good many hundred hours of stop-watch readings on mixer operation. Table 3 gives the stop-watch readings on an ordinary hour's production; and Table 4 gives the summary of seven successive studies made on this job and indicates the efficiency maintained during ordinary operation.

Table 2.—*Stop-watch study of a perfect hour's operation on concrete pavement construction, Missouri Federal-aid project 228* [1]

Batch No.	Mixing time: Charge	Mixing time: Mix	Mixing time: Discharge	Batch No.	Mixing time: Charge	Mixing time: Mix	Mixing time: Discharge
	Minutes	*Minutes*	*Minutes*		*Minutes*	*Minutes*	*Minutes*
1	0.16	1.04	0.06	27	0.16	1.03	0.06
2	.16	1.04	.06	28	.16	1.02	.04
3	.16	1.05	.06	29	.16	1.04	.04
4	.16	1.02	.04	30	.16	1.04	.06
5	.16	1.04	.04	31	.16	1.04	.08
6	.16	1.04	.05	32	.16	1.06	.04
7	.16	1.05	.03	33	.16	1.03	.06
8	.16	1.02	.05	34	.16	1.02	.06
9	.16	1.03	.06	35	.16	1.04	.05
10	.16	1.04	.03	36	.16	1.03	.04
11	.16	1.04	.06	37	.16	1.06	.05
12	.16	1.03	.04	38	.16	1,05	.07
13	.16	1.04	.06	39	.16	1.03	.05
14	.16	1.02	.04	40	.16	1.03	.05
15	.16	1.04	.06	41	.16	1.03	.06
16	.16	1.02	.09	42	.16	1.06	.06
17	.16	1.04	.05	43	.16	1.04	.08
18	.16	1.02	.05	44	.16	1.04	.04
19	.16	1.04	.05	45	.16	1.03	.04
20	.16	1.04	.04	46	.16	1.04	.07
21	.16	1,04	.05	47	.16	1.05	.05
22	.16	1.04	.04	48	.16	1.06	.09
23	.16	1.01	.04				
24	.16	1.05	.05	Total	7.68	49.79	2.53
25	.16	1.03	.04	Average	.16	1.04	.05
26	.16	1.05	.05	Percentage	13	83	4

During this hour's operation 48 batches were produced—the maximum possible number under the specifications. There was no lost time.

This record is published because it is a practical demonstration of the fact that very high efficiency can be maintained on paving work if the job is properly organized and energetically administered. It may be of interest to add that while the stop-watch studies reported in Tables 2 and 3 were taken by representatives of the Bureau of Public Roads, the use of the stop watch was not new to the superintendent of this job and the studies revealed nothing that he had not learned before the bureau's representatives arrived. The job was not only well organized but its organization was based on a thoroughgoing conception of the principles of scientific management, which have been discussed in previous articles on concrete construction work published in Public Roads.

Table 3.—*Stop-watch study of an ordinary hour's operation on concrete pavement construction, Missouri Federal-aid project 228*

Batch No.	Mixing time: Charge	Mixing time: Mix	Mixing time: Discharge	Lost time: Mixer trouble	Lost time: Truck shortage	Lost time: Water-supply trouble
	Minutes	*Minutes*	*Minutes*	*Minutes*	*Minutes*	*Minutes*
1	0.16	1.03	0.10			
2	.16	1.07	.08			
3	.16	1.02	.06			
4	.16	1.05	.07			
5	.16	1.03	.08			
6	.16	1.01	.10	.08		
7	.16	1.02	.06			
8	.16	1.05	.10		.02	
9	.16	1.03	.07			
10	.16	.96	.06			
11	.16	1.01	.06			
12	.16	1.02	.06			
13	.16	1.02	.07			
14	.16	1.00	.03			
15	.16	1.04	.05			
16	.16	1.03	.06			
17	.16	1.01	.04			
18	.16	.96	.05			
19	.16	1.03	.07			
20	.16	1.04	.07			
21	.16	1.01	.06			
22	.16	1.01	.05			
23	.16	1.04	.10		1.86	
24	.16	1.05	.06			
25	.16	1.01	.06			
26	.16	.97	.05			
27	.16	1.01	.07			
28	.16	1.00	.07			
29	.16	1.03	.07			
30	.16	.96	.08			
31	.16	1.05	.10		.10	
32	.16	1.06	.04			
33	.16	1.00	.05			
34	.16	1.01	.08			
35	.16	1.03	.10		.04	
36	.16	1.02	.10		.06	
37	.16	1.05	.10			1.24
38	.16	1.06	.10			
39	.16	1.06	.08			
40	.16	1.07	.07			
41	.16	1.04	.07			
42	.16	1.06	.10	.10		
43	.16	1.04	.09			
44	.16	1.02	.06			
45	.16	1.02	.10			
Total	7.20	46.11	3.19	.18	2.08	1.24
Average	.16	1.02	.07			
Percentage	12	77	5½		3½	2

Table 4.—*Summary of seven stop-watch studies on Missouri Federal-aid project 228*

Study No.	Number of batches per hour	Efficiency of outfit column 2 divided by 48	Mixer time: Charge	Mixer time: Mix	Mixer time: Discharge	Mixer time: Total	Lost time: Truck delays	Lost time: Water-supply trouble
		Per cent	*Seconds*	*Seconds*	*Seconds*	*Per cent*	*Seconds*	*Seconds*
1	44	91.7	9.6	62.4	3.0	92	8	
2	44	91.7	9.6	61.8	3.0	91	9	
3	45	95.8	9.6	61.2	4.2	94½	3½	2
4	44	91.7	9.6	61.8	4.2	92	8	
5	42	87.5	9.6	62.4	4.2	89½	10½	
6	48	100.0	9.6	62.4	3.0	100		
7	45	95.8	9.6	61.2	4.2	93¾	6¼	

MODERN HIGHWAY TRAFFIC AND THE PLANNING OF STATE HIGHWAY SYSTEMS

Reported by Dr. J. G. McKAY, Chief, Division of Highway Economics, United States Bureau of Public Roads

The last 15 years have seen the reemergence of highway transportation, one of the oldest methods for the movement of people and goods, from the position of comparative unimportance to which it had fallen during the period of greatest railroad development to a position of the first rank in the national scheme of transportation. Mass movement of people and commodities on the principal routes of the various State highway systems confronts us as a fact and not a theory.

The general field of motor-vehicle transportation can be divided into three major classes of service. First is the local distribution of commodities and local transportation of people. This service constitutes in tonnage the bulk of the motor-truck movement and is primarily the distribution of goods within cities and their suburban areas. In Connecticut 67.1 per cent; in Ohio 64.2 per cent; and in Pennsylvania 77.3 per cent of the net tonnage transported by motor truck is hauled less than 30 miles. Seventy per cent of the net tonnage hauled in Cook County by motor truck is a direct distribution of commodities to points of final use. In passenger transportation the principal function is also mass transportation within local areas.

The second principal class of motor-vehicle service supplements existing rail and water service by extension of freight and passenger service into areas not served by rail or water lines; substitutes motor-vehicle service for rail operation of unprofitable branch lines; and provides a combined service in conjunction with railroads or boat lines or both. The primary function of the motor vehicle in this joint movement is the movement of people or goods in the short haul.

The third class of motor-vehicle service is the so-called long-haul transportation. This type of service is not important as to quantity of movement nor would it appear to be economically sound. For motor trucks, at least, the volume of tonnage in the long-haul zone is comparatively small and decreasing in importance. In Connecticut 15.2 per cent; in Pennsylvania 6.9 per cent; and in Ohio 15.9 per cent of the net tonnage is hauled over 60 miles. Usually long-haul transportation is limited to movements in which speed of delivery or some special characteristic is the principal determining factor.

A relatively small percentage of motor trucks are engaged in the commercial transportation of freight. On the Maine highways 8.7 per cent; on the Pennsylvania highways 13.6 per cent; and on the Ohio highways 21 per cent of the loaded trucks are commercial, operating either for hire or on contract.

Analysis of motor truck transportation in the several States shows that the development of a large system of motor trucking over a considerable area beyond the short-haul is economically unsound. The movement out of cities is primarily a local process of distribution. The movement toward cities consists largely of goods produced in the area, principally food products. The volume of goods moving between two shipping points is unbalanced in the short-haul and progressively unequal with increase in distance. This fact negatives the development of long-haul motor truck transportation as a system of haulage.

The cost of providing motor-truck common-carrier service increases with increase in distance, owing primarily to the increasing terminal cost ratio to total net revenue, return movement of empty trucks, and partial loads at full rates or full loads at low rates.

The principal function of the motor vehicle as part of a correlated system of rail, water, and highway transportation, as indicated by present trends, seems to be its development in the short-haul zone with great potential possibilities of volume service in terminal areas of dense population.

The functional relationship of the motor vehicle and water transportation would appear to be the transfer of goods to and from docks and the mechanical organization of motor truck loading and unloading methods in the water shipping area to facilitate rapid loading and unloading of cargoes.

The principal service of the motor truck as a part of organized rail transportation is primarily in the rail freight transfer areas and in the short-haul movement of rail package freight. Whatever we may think of present motor vehicle operating methods, if there is a real demand for this type of service it will continue to develop. The problem lies in the intelligent organization, improvement and control of this new type of transportation. Probably the next few years will see the organization of large motor vehicle operating companies functioning as a part of existing rail and water systems or as independent operating companies. Since the haulage of goods and people can be generally safeguarded by regulatory legislation, it would seem that present rail or water lines should qualify as the responsible financial and experienced agencies of mass transportation of people and goods in this rapidly developing field of motor vehicle transportation.

Meanwhile, no matter under whose guidance the organization of motor vehicle operation is developed, there remains the present problem of intelligently planning highway systems to serve this rapidly growing method of transportation.

PLANNING THE IMPROVEMENT OF HIGHWAY SYSTEMS

There is no fundamental difference in principle between the public business of developing systems of highways, and private enterprises engaged in producing commodities or in the performance of services. For example, light and power, gas and telephone utilities and other industries are all engaged in the production of their commodities for public use. The history of their modern development and expansion is largely a development based upon a careful analysis of the demand for their product by present and potential consumers in a given area. The soundness of their analysis of the need for new service and the expansion of their plant anticipating the demand for their product has been an influential factor in the progress or lack of progress of many communities.

The same basic economic and engineering principles of management that exert such a controlling influence in the field of private business should govern the public business of production in the highway field.

Applied to the public business of a State responsible for developing a connected system of improved highways to facilitate the transportation of people and commodities, the first basic principle of production management is that the various sections of a highway system selected for improvement and the type of improvement chosen for each section should be based upon present and expected future traffic demands, modified by the various physical and economic characteristics which affect the choice of specific construction types to be built on the various sections of a State highway system.

The second principle is the familiar one of the budget upon which all financially sound industries operate. Applied to the highway business it involves; (1) the determination of the amount of money required to complete the improvement; (2) the apportionment of the cost among those who benefit from the improvement of the highways, and provision through necessary legislation for raising the required highway funds. This management must insure the expenditure of the money in accordance with a predetermined plan of improvement in which each route or section of a route is to be improved to the degree required by the traffic and to no greater or less degree. The raising of revenue is the responsibility of the State legislature, and is always the final limiting factor in any program of public works. Provision by a legislature of more than the necessary revenue is apt to encourage waste; provision of sufficient funds, well managed by the highway department, results in a well-balanced system of highway improvements and the economic development of the State as a whole; provision of insufficient improvement funds defers the true improvement of a highway system, forces the highway department to spread uneconomically its expenditures of State funds over the entire State system, usually results in the development of a highway system below the requirements of traffic, and, if continued over a period of years, increases the total ultimate cost of highway transportation.

The establishment of scientific plans for highway development, which will result in the maximum of highway improvement and highway transportation service, with available revenue, labor, equipment, and building materials, requires a careful analysis of highway traffic, the trend of its development and its distribution over the highway system. The necessity of such an analysis is now recognized by highway executives, but their efforts have been handicapped by lack of precise knowledge of the character and amount of present and expected future traffic using the various sections of the system.

The plan of State highway improvement may materially alter the economic and social development of a people as a whole or any section thereof. The location and improvement or lack of improvement of a given route is of vital importance not only to the traffic of the immediate locality but also to the traffic of larger areas. Therefore, the development of a system of highways should not be judged as miles and types of highways constructed each year but considered in terms of the movement of people and goods. The planning and construction of a connected system of highways deal in fact with the destiny of localities and States, their agriculture, their industries, the growth of suburban areas adjacent to centers of population, and the social activities of a people. This is a tremendous social responsibility and not merely a problem of physics concerning mixtures of cement, water, brick, steel, bitumens, stone aggregate, gravel, equipment, and labor into what we now term the modern road.

WHETHER TO IMPROVE AND HOW MUCH THE FIRST QUESTIONS

The major problem is not one of the particular type of materials to use, but rather whether to build or not, and how much highway service should be furnished in a given area. Upon the proper solution of these problems depend the well-being and progress of a people. Considering the improvement of highways from this point of view there can be no question concerning the necessity of developing sound plans for highway improvement over a period of years in the several States, and of providing the necessary money to carry out economically the proposed plan of improvement.

The principal classes of highway improvements are: (1) New construction, (2) stage construction, (3) reconstruction, (4) building of bridges and culverts, (5) highway and railroad grade separations, (6) widening of present highways, and (7) methods of guiding and safeguarding traffic.

Each of these classes of improvement, although more or less distinct as a class, is part of a general scheme of betterment, and especially within congested population and traffic areas the highway engineer faces the urgent need of solving all these complicated problems of highway improvement. The development of the plan as a whole, including each of the several classes of improvement, should be based largely upon present and expected future traffic and "lay-out" and condition of the existing highway system in any given area.

The first step in planning a program of highway improvement is the measurement of the present, and the prediction of the future volume and character of traffic on the State primary, secondary and tertiary systems. The principal traffic factors involved in judging the relative traffic importance of the three systems, or sections of each system, are the average daily and maximum total traffic, and the average daily and maximum truck traffic using each section. The average daily number of loaded light (one-half to 2½-ton), medium (3 to 4-ton) and heavy (5 to 7½-ton) vehicles is an important factor in the determination of the plan of improvement as well as in the selection of the types to be constructed.

The second step is the determination of the relationship between population and demands for highway service and the consideration of present density of population and population trends as an aid in the development of a plan of improvement which will most efficiently serve the traffic needs of this population.

The next step is the classification of the various highways or sections of highways as major traffic routes (class A), secondary traffic routes (class B), and minor traffic routes (class C). A class A highway is defined as one that requires one of the so-called rigid types of improvement, concrete, brick, bituminous concrete, or their equivalent. A class B highway is defined as one that requires a so-called flexible type of improvement, such as standard bituminous penetration macadam or its equivalent. A class C highway is defined as one that requires other lesser types of improvement.

The principal traffic factors involved in such a classification are:

(*a*) Average daily and maximum total traffic and truck traffic.

(*b*) Forecast of average daily total traffic and truck traffic for periods of 5 and 10 years.

(*c*) Average daily and expected future number of loaded light, medium, and heavy trucks for each route or section of route.

(*d*) The ratio of the total number loaded trucks to the total traffic in order to separate for special consideration routes or sections of routes on which motor trucks are an abnormally large or small proportion of total traffic.

(*e*) The number and frequency of critical heavy loads.

(*f*) Average maximum traffic as one measure of the width of the improvement, the necessity for improvement of additional parallel routes and the "by-passing" of congested centers of local traffic.

(*g*) Analysis of highway maintenance and capital costs and vehicle operating costs as an important factor in determining the traffic limits for the various types of improvement.

The fourth step is the measurement of motor-vehicle mileage on the primary, secondary, and tertiary highway systems, and the estimate of the earning capacity of these three systems to determine the relative vehicle-use value of each as a guide in developing the plan of improvement and the budgeting of construction and maintenance funds.

Finally, we must have a thorough analysis of the present system and the physical condition of the existing improvements on it, since the plan of betterment must, in general, incorporate the existing State highway system as the basis of the improvement plan.

SPECIAL CONSIDERATION NECESSARY FOR HIGHWAY SYSTEM ADJACENT TO CITIES

A State plan of highway improvement can be separated into two distinct planning phases.

The first is the general State plan, consisting of a connected system of primary, secondary, and tertiary routes serving each section of the State. It should be recognized that just as there is a considerable variation in the present and expected future volume of traffic on the highway systems of the different States, so there is within each State also considerable variation in the present and expected future traffic on the various sections of the primary and secondary system.

The second phase of the State plan is the special consideration necessary in areas adjacent to centers of population. The improvement plan of the State and the plan of improvement of the larger cities within it should be worked out cooperatively.

This cooperative planning is essential to the proper location and entry of State routes into congested traffic areas, to avoid dumping traffic from one or more than one State route into an already congested area, to provide for adequate connections and improvement of the city streets that join State routes at city limits, to make provision for "by-passing" congested traffic areas, to eliminate obstructions to the easy movement of traffic, and finally to provide belt, arterial, and secondary local traffic routes to facilitate the rapid, safe, and unobstructed flow of traffic in congested traffic areas.

In the final analysis the worth of a transportation survey and the resulting plan of highway improvement is measured by the actual highway construction, reconstruction and widening program which is carried into effect over a period of years.

The State highway engineer, as the executive director of the public business of providing highways, is responsible (1) for the analysis of the traffic demand for his product on the various sections of the State system; (2) for a financial analysis of the yearly cost, the revenues required, the funds available, and for the establishment of a budget for the period of the improvement program; and (3) for the business and engineering management of the improvement program.

The major limiting factor is the financial program set up by the legislative organization responsible for raising the revenue to give reality to any plan of highway improvement; and therefore a large part of the responsibility for the character and condition of a State system of highways rests upon the department of the State government responsible for the raising of highway funds and not upon the department charged with the duty of constructing the highways.

(Continued from p. 184.)

One illustration is to the point. It so happens that the writer was engaged in 1909 in building 12 miles of branch-line railroad from Carthage to Pinehurst to connect with a main line at this latter point. There was a buggy factory at Carthage as well as two or three other factories, then served by a branch line 10 miles long from one of the large trunk-line railroads.

The second line was built to compete with the first for the output of the town's factories and for the hauling to it of coal, raw material, and supplies which service incidentally afforded the town relief from high rates prevailing on the other branch line.

The town was thus situated in a general way at the apex of a triangle composed of these branch lines on two sides and the main lines on the third. Neither of these branches made much money serving as feeders to the main line and as time went on handled less and less business. The interests which had in 1911 bought the branch from Pinehurst to Carthage tore up the tracks about 1914. In his connection with the State Highway Commission 10 years later, the writer took part in building a State highway on the grade of the branch rail line he had built in 1909.

To-day the products of the buggy factory move by highway either from Carthage to the rail station at Pinehurst or over another State highway by truck to Sanford. The remaining branch line into Carthage was sold some time ago to a group of local people who wished its operations continued, but its business has declined almost to the vanishing point and it is probably only a question of time until it will be abandoned. Its business is largely in the moving of heavy, bulk freight such as coal, sand, and gravel.

Our problem in the highway field when and where called on to provide adequate highways to replace these branch lines will be the finding of means to move these heavy commodities longer distances over the highways. Other tonnage of many of these lines has either gone or is going more and more rapidly to the highways because of the generally short distances involved and the small aggregate total of such tonnage.

The data furnished might be multiplied ad infinitum but these illustrations will doubtless serve to indicate the important transportation service now rendered the State by our highway system. It serves the other transportation agencies equally as well as it serves industry, agriculture and other groups.

New fields of industry, and agriculture are being opened, and the old fields are being stimulated immeasurably by the highway. It goes without saying that all this assures to the rail carriers much business from the South and proportionate prosperity.

THE USE OF HIRING CARS AND BUSSES ON RURAL HIGHWAYS

Reported by **HENRY R. TRUMBOWER**, Economist, United States Bureau of Public Roads

According to the report of the Commissioner of Internal Revenue, 165,372 motor vehicles were engaged in the business of carrying passengers for hire on June 30, 1925. The internal revenue law provides that owners of automobiles for hire with a capacity of from two to seven passengers shall pay a yearly excise tax of $10 to the Federal Government, and those owning automobiles used to carry passengers for hire which have a capacity of eight and over are obliged to pay an annual tax of $20. For the year ended June 30, 1925, the Federal Government collected in this manner $1,838,079 which comprised the taxes upon 141,849 cars of small capacity, and 23,523 cars of large capacity, the latter being largely of the motor-bus type.

These figures show that 85 per cent of the passenger automobiles used for hire in the United States are small cars with a capacity of from two to seven passengers. The cars seating more than seven passengers constituted only 15 per cent of the total number of hiring cars. The taxicabs used in cities account for by far the largest number of cars of the smaller passenger capacity.

The proprietors of the cars paying this tax numbered 96,475. When this number is compared with the total number of hiring cars it is seen that the ownership averages but 1.5 cars of both types, large and small, per owner or proprietor. In spite of the fleets of taxicabs owned by one individual or by a company the average number of cars owned by an operator is less than two.

The ratio of passenger cars for hire to the total population of the country is one car for every 686 people. At the close of 1924 there were 15,460,649 passenger automobiles in the United States. For every passenger car used for hire, therefore, there were then approximately 90 passenger cars used by their owners as private cars.

The ratio of the number of hiring passenger cars to the population differs in the several States and also in the various sections of the country. In Table 1 there are shown for the various groups of States the total number of passenger cars for hire, the estimated population, and the number of persons per passenger car used for hire.

Table 1.—*Number of hiring cars and relation to population by groups of States*

Group of States	Total number of passenger cars for hire	Population	Number of persons per car
New England	17, 840	7, 883, 449	442
Middle Atlantic	44, 272	23, 929, 700	540
East North Central	30, 807	23, 302, 538	756
West North Central	15, 016	13, 057, 696	869
South Atlantic	17, 972	15, 007, 396	835
East South Central	10, 364	9, 170, 847	885
West South Central	13, 195	11, 068, 039	840
Mountain	6, 426	3, 736, 666	580
Pacific	9, 480	6, 345, 595	669
Total	165, 372	113, 501, 926	686

For these several groups of States the variation in the number of persons per car ranges from 442 in the New England States to 866 in the West North Central States, the average for the whole country being 686. At first glance it would appear as if the number of persons per car were smaller in those States where the population is most dense. That holds true for the New England and the Middle Atlantic group of States, which contain many and large cities. In such cities the number of taxicabs operated is naturally large. On the other hand, in the Mountain States the number of persons per car is 580, just slightly higher than in the Middle Atlantic States, where the ratio of persons to cars is 540. In these Mountain States this ratio is less than in any one of the other groups of States except the New England and the Middle Atlantic groups.

RATIO OF HIRING CARS TO POPULATION WIDELY DIFFERENT IN VARIOUS STATES

In the individual States a still greater variation in persons per car is found. In New Hampshire, for example, there are 218 persons for every passenger automobile used for hire; that is the lowest ratio found anywhere. The highest is found in North Dakota, where there are 1,690 persons per car. The rest of the States are found between these two extreme ratios. In Table 2 the States are classified according to the number of persons per car.

Table 2.—*Classification of States according to number of persons per hiring car*

NUMBER OF PERSONS PER HIRING CAR

200	300	400	500	600
New Hampshire. Vermont.	Colorado. Maine. New York. Wyoming. District of Columbia.	Florida. New Jersey. Rhode Island.	Connecticut. Maryland. Massachusetts. Nevada.	California. Illinois. Indiana. Missouri.
700	800	900	1,000	1,100
Arizona. Idaho. Kansas. Kentucky. Texas.	Alabama. Arkansas. Michigan. Oregon. Virginia. Washington.	Delaware. Iowa. Montana. New Mexico. Ohio. Oklahoma. Tennessee. West Virginia. Wisconsin.	Minnesota. Mississippi. Nebraska. Pennsylvania. South Carolina.	Louisiana. North Carolina.
1,200	1,300	1,400	1,500	1,600
Utah.	South Dakota.	Georgia.		North Dakota.

Examination of this summary discloses no particular reason for the variation in the distribution of the hiring cars. In States as widely different in density of population as New York and Wyoming and Nevada and Massachusetts, the ratio between passenger cars used for hire and population is approximately the same; New York has 396 persons per car, Wyoming has 353; Nevada has 540 persons per car and Massachusetts 555. In Georgia the ratio is 1,440 persons per car and in the adjoining State of Alabama it is only 872 persons per car. Nevertheless the density of population in these two States differs very slightly; in Alabama it is 45.8 persons per square mile and in Georgia 49.3. The density of population does not appear to be the controlling factor as to the per capita use made of passenger cars for hire. To a certain extent it may be said that the extent to which the hiring cars are used depends upon the distribution of the population of a State between urban and rural communities. In the New England States 79.2 per cent of the population is urban; in the Middle Atlantic States the corresponding ratio is 74.9 per cent. In those two groups of States is found the smallest number of persons per passenger car used for hire. The Mountain States, however, appear to be an exception to this rule. There the urban population amounts to but 36.4 per cent of the total population while the ratio of number of persons per car is 580, the third lowest of the different groups of States. The lack of sufficient transportation facilities in this group of States may account for the relatively greater development of the transportation of passengers by common-carrier automobiles.

The proportion of small-capacity automobiles engaged in the transportation of passengers for hire also differs in the several groups of States. Table 3 shows the number and percentage relationship of the 2 to 7 passenger cars and the cars with a passenger capacity of 8 and over.

TABLE 3.—*Number of small and large capacity hiring cars by groups of States*

Group of States	2 to 7 passenger cars		8 passengers and over		Total number of hiring cars	
	Number	*Per cent*	*Number*	*Per cent*	*Number*	*Per cent*
New England	15,492	87	2,348	13	17,840	100
Middle Atlantic	38,383	86	5,889	14	44,272	100
East North Central	25,236	82	5,571	18	30,807	100
West North Central	13,161	88	1,855	12	15,016	100
South Atlantic	15,566	87	2,406	13	17,972	100
East South Central	9,547	92	817	8	10,364	100
West South Central	12,131	92	1,064	8	13,195	100
Mountain	5,206	81	1,220	19	6,426	100
Pacific	7,127	75	2,353	25	9,480	100
Total	141,849	85	23,523	15	165,372	100

The small-capacity car is found to be predominant in each of the geographic sections, the average for the country being 85 per cent. In the various groups of States it ranges from 75 per cent in the Pacific States to 92 per cent in the East and West South Central States. These relationships differ still more widely in the individual States. In the individual States the cars of large capacity range from 4 per cent of the total number in Nevada to 45 per cent in Wyoming. The States are classified in the following tabulation, Table 4, according to the ratio which the large-capacity cars bear to the total number of cars used for hire.

TABLE 4.—*Classification of States according to ratio of large-capacity hiring cars to the total number of hiring cars*

Ratio of number of large-capacity hiring cars to total number				
0 to 9 per cent	10 to 19 per cent	20 to 29 per cent	30 to 39 per cent	40 to 49 per cent
Alabama.	Arizona.	California.	Michigan.	Wyoming.
Arkansas.	Colorado.	Delaware.	New Jersey.	
Georgia.	Connecticut.	Indiana.	Washington.	
Louisiana.	Florida.	Maryland.		
Maine.	Idaho.	Minnesota.		
Mississippi.	Illinois.	Ohio.		
Nebraska.	Iowa.	Oregon.		
Nevada.	Kansas.	Utah.		
New Hampshire	Kentucky.			
New Mexico.	Massachusetts.			
New York.	Missouri.			
North Dakota.	Montana.			
Oklahoma.	North Carolina.			
Rhode Island.	Pennsylvania.			
South Carolina.	South Dakota.			
Tennessee.	Vermont.			
Texas.	Virginia.			
West Virginia.	Wisconsin.			
	District of Columbia.			

In 12 of the States the large-capacity cars used for hire amount to over 20 per cent of the total number of cars used as common carrier passenger cars. The extent to which these large-capacity cars or busses are used depends upon the development of bus service within cities and upon the rural highways. In New Jersey, for instance, where 30 per cent of the passenger cars for hire are of the large-capacity or bus type, a very extensive motor-bus service has been developed in the urban and suburban sections of the northern part of the State where there are a number of large cities and where the population is very dense. It may be said that in that case the bus service is primarily urban and suburban. In Washington it is found that 32 per cent of all of the passenger cars for hire are of the large-capacity type and 68 per cent have a capacity of from 2 to 7 passengers. This comparatively large number of large-capacity cars is very largely due to the development of interurban bus service. In Washington that type of service is very extensive.

NUMBER OF HIRING CARS USED ON RURAL ROADS ESTIMATED

These data from the reports of the internal revenue collectors furnish no clews with respect to the use of these passenger cars for hire within cities and towns on the one hand and in interurban or suburban service on the other. From facts which have been ascertained relative to the number of motor busses operating over the rural highways of certain States it is possible to make an estimate of the number of the large-capacity cars for hire which are operated in interurban and suburban service. In seven of the States it was found that there were 823 common carrier passenger vehicles operating on regular routes over rural highways and outside of towns and cities.[1] All of these had a capacity of eight passengers and over. In these same States the internal revenue collector's reports show that there were in all 2,442 of the large-capacity cars. This would indicate that about a third of the large type of cars are used on interurban and suburban routes, the rest in service on city and town streets. According to these figures it is perhaps not far from the facts to say that approximately 8,000 of the 23,523 large-type motor cars used for hire are operating in interurban and suburban service.

A very much smaller proportion of the small type of cars, seating from one to seven passengers, is found on the rural highways. In the same seven States referred to above [2] it was found that there were only 507 small-type passenger cars engaged in common carrier service on interurban and suburban routes. The Internal Revenue collector's reports show that there were in all 12,677 of these small-type cars in the same States. It follows, therefore, that approximately 4 per cent of the small-type cars were operated over rural highways and the remaining 96 per cent or 12,170 cars were engaged in urban service. If this ratio is applied to the total of 141,849 small-type cars reported by the Internal Revenue Department it follows that about 5,670 cars of this type are in rural common carrier service.

Upon the basis of these estimates it may be said that out of a total of 165,372 passenger cars, consisting of both large and small types used for hiring purposes, about 13,670, or about 8 per cent, are operated over interurban and suburban routes.

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[1] TRUMBOWER, HENRY R., THE MOTOR BUS AS A COMMON CARRIER, Public Roads, p. 213, vol. 6, No. 10, Dec. 1925.

[2] Connecticut, New Hampshire, West Virginia, Kentucky, Arizona, Washington, and Maryland.

THE PENETRATION OF WOOD PRESERVATIVES

Reported by **ROBERT I. RUDOLPH** United States Bureau of Public Roads

The specifications of the American Association of State Highway Officials for timber highway bridges require that the heads of the treated timber piles after having been cut to receive caps and prior to the placing of caps shall be treated to prevent decay. Two methods of treatment are specified. The first consists of three applications of a mixture composed of 60 per cent creosote oil and 40 per cent tar roofing pitch, and the second consists of three treatments with hot creosote oil after which the portion so treated is covered with hot roofing pitch. Tests have been made by the Bureau of Public Roads to determine the relative value of such treatments as to the depth of penetration into the wood.

Because of difficulty in procuring treated timber pile heads, the tests were made on wood blocks treated as specified by each of the two methods.

The wood blocks used for the purpose were approximately 6 by 6 by 12 inches in size and were made from a commercially known long-leaf, air-dried, dressed, Georgia yellow pine.

The analysis of the tar roofing pitch was as follows:

Specific gravity at 25°/25° C		1. 264
Softening point		55° C.
Bitumen soluble in CS_2	per cent	74. 86
Organic matter insoluble	do	24. 98
Inorganic matter insoluble	do	0. 16
Ductility at 25° C., centimeters		100+
Distillate up to 300° C	per cent	4. 49
Residue	do	95. 31
Water		Trace.
Specific gravity of distillate at 38°/15.5° C		1. 043

The creosote oil showed the following analysis:

Specific gravity at 38°/15.5° C		1. 062
Insoluble in benzol	per cent	. 03
Coke	do	5. 33
Water		None.
Distillate to 210° C	per cent	0. 70
210°-235° C	do	8. 66
235°-270° C	do	26. 43
270°-315° C	do	27. 39
315°-355° C	do	20. 43
Residue	do	16. 00
Specific gravity of distillate (235°-315° C.) at 38°/15.5° C		1. 028
Specific gravity of distillate (315°-355° C.) at 38°/15.5° C		1. 084
Float test of residue at 70° C	seconds	131

Mixture of 60 per cent creosote oil and 40 per cent tar roofing pitch:

Specific viscosity at 40° C	1.03

Before beginning the work it was decided to broaden the scope of the tests by treating separate groups of blocks by each of the two methods specified with cold and hot applications of preservative. Thus by the first method a group of blocks was treated with a cold mixture of creosote oil and tar and another group with a hot mixture of creosote oil and tar. By the second method one group was treated with cold creosote oil and another with hot creosote oil and the specimens of both of the latter groups were covered with hot roofing pitch. Each of the groups treated with preservative consisted of three wood blocks.

The preservative was thoroughly brushed on the end of the block and allowed to dry. The material applied hot was heated to a temperature of 70° C. The first application was fairly dry after one and one-half hours and a second treatment was given. This coat took somewhat longer to dry and the third treatment was applied at the end of three hours. It was noticeable at the time of treatment that the creosote-oil applications seemed to be absorbed more readily than the creosote-oil and tar mixture. Tar heated to a temperature of 105° C. was applied two hours after the third application of creosote oil. The tar did not adhere readily to the creosoted surface and could not be brushed on. Therefore the hot tar was poured upon the treated ends and the entire surface was covered and smoothed over by means of a hot knife.

The tops of the blocks were sawed off 2 inches below the treated ends. These tops were then split at different sections and the average distance of penetration measured. The results of these measurements are given in Table 1.

TABLE 1.—*Average penetration of preservatives applied*

Treated with creosote oil and tar mixture (cold)		Treated with creosote oil and tar mixture (hot)		Treated with creosote oil (cold), covered with tar (hot)		Treated with creosote oil (hot), covered with tar (hot)	
Block	Average penetration	Block	Average penetration	Block	Average penetration	Block	Penetration
	Inches		*Inches*		*Inches*		*Inches*
1	1/8	1	1/8	1	13/64	1	14/64
2	1/8	2	1/8	2	13/64	2	16/64
3	1/8	3	1/8	3	10/64	3	18/64
Average	1/8	Average	1/8	Average	3/16	Average	1/4

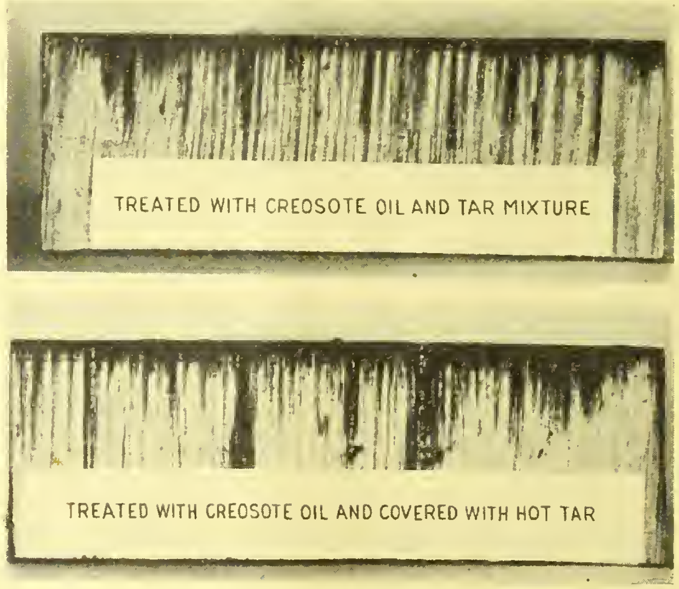
TREATED WITH CREOSOTE OIL AND TAR MIXTURE
TREATED WITH CREOSOTE OIL AND COVERED WITH HOT TAR

FIG. 1.—COMPARISON OF THE DEPTH OF PENETRATION OBTAINED ON REPRESENTATIVE BLOCKS BY THE TWO METHODS OF TREATMENT

Figure 1 shows the depth of penetration obtained on representative blocks by the two methods of treatment.

Nearly all of the wood blocks had a small portion of sapwood at one of the corners. The blocks treated with creosote oil, either cold or hot, showed that the creosote oil had penetrated this sapwood for a distance in excess of 2 inches. This was not so evident in the blocks treated with the mixture of creosote oil and tar.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
*1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94C. TNT as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60M. Federal Legislation Providing for Federal Aid in Highway Construction.
*62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects. 5c.

FARMERS' BULLETINS

No. *338F. Macadam Roads. 5c.
*505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *739Y. Federal Aid to Highways, 1917. 5c.
*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPTEMBER 30, 1926

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,725,986.09	1,298.3	$ 1,033,288.73	$ 492,916.35	75.7	$ 5,938,997.16	$ 2,809,565.96	882.4	$ 141,817.86	$ 70,908.93	13.4	$ 2,260,078.67	Alabama
Arizona	10,949,878.25	5,863,772.35	729.8	272,263.37	172,004.74	26.7	1,480,029.04	1,018,028.97	80.9				2,563,442.94	Arizona
Arkansas	18,364,544.50	7,666,698.35	1,323.0	535,286.95	262,461.46	38.9	4,412,266.23	2,089,640.28	284.6	1,169,443.90	580,750.55	89.4	1,006,253.36	Arkansas
California	27,142,596.90	13,003,592.30	1,058.0	2,962,808.88	1,471,835.86	105.7	10,051,011.39	4,916,686.49	239.8	351,129.85	199,807.86	5.6	2,480,892.49	California
Colorado	13,905,904.64	7,127,288.18	745.0	734,611.99	398,706.82	29.1	5,521,006.50	2,666,676.46	245.7	367,491.95	199,837.34	24.3	1,933,303.20	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1	415,786.78	153,899.62	8.0	4,831,765.69	1,314,166.86	60.6	1,015,854.12	281,827.61	14.8	483,201.11	Connecticut
Delaware	4,918,052.29	1,781,665.60	124.3	672,626.55	275,874.75	17.6	.769,795.50	334,016.25	17.9	243,288.10	71,705.65	8.3	10,795.85	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	2,111,145.69	1,049,833.65	62.7	8,378,324.74	3,785,525.05	227.2	430,300.34	167,816.51	1.9	1,257,416.47	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	1,998,849.78	981,417.94	129.4	11,764,126.92	5,706,114.50	536.6	31,696.14	15,848.05	7.6	64,334.65	Georgia
Idaho	11,061,198.14	5,882,112.70	724.7	559,674.29	277,975.81	18.9	2,951,067.43	1,808,258.53	195.2	599,657.75	345,287.73	60.4	245,992.23	Idaho
Illinois	44,116,611.86	20,619,995.74	1,377.7	2,012,307.84	980,960.66	68.1	9,084,347.31	4,413,190.03	328.7	909,576.93	428,978.44	30.6	3,389,073.13	Illinois
Indiana	16,949,425.87	8,172,125.19	534.3	2,408,698.84	1,135,591.61	71.0	17,470,671.26	8,223,878.57	502.5				672,759.63	Indiana
Iowa	29,062,375.40	11,926,302.10	2,114.8	1,963,084.70	911,665.80	153.3	12,986,351.76	5,744,179.50	663.2	1,719,562.61	601,968.35	76.0	301,447.25	Iowa
Kansas	32,826,601.64	12,590,489.25	1,160.6	522,940.32	235,898.35	48.9	13,455,886.18	5,236,974.47	745.6	2,925,031.07	1,027,403.41	126.0	373,645.52	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	350,565.12	126,381.50	9.1	7,930,690.06	3,716,021.58	382.8	564,016.62	282,008.29	33.5	596,315.38	Kentucky
Louisiana	13,830,592.68	6,144,739.99	1,055.9	394,175.75	192,034.39	18.7	3,839,666.52	1,861,498.23	188.6	1,440,684.19	621,640.40	45.7	452,494.99	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6	454,430.00	189,894.49	16.6	2,676,213.43	952,195.53	73.4	1,084,926.14	402,821.22	33.2	727,409.37	Maine
Maryland	10,924,943.10	5,112,991.22	423.3				1,350,356.90	641,523.57	72.2	426,154.66	170,540.72	16.2	1.49	Maryland
Massachusetts	18,353,757.71	6,657,260.82	374.6	342,513.05	87,146.25	4.5	4,939,295.15	1,356,464.15	69.2	773,451.02	199,013.17	11.9	1,808,841.81	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	390,946.77	188,717.04	13.6	13,675,367.82	6,227,830.93	397.4	536,564.96	190,152.69	7.8	1,908,612.04	Michigan
Minnesota	37,170,985.95	15,586,116.56	3,181.9	2,893,342.12	1,304,359.07	250.6	7,011,319.45	2,583,500.00	340.6	2,745,520.32	49,000.00	128.7	68,804.37	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1,129.0	821,126.41	407,936.55	49.9	7,847,680.14	3,831,403.27	396.8	384,133.70	185,185.26	37.6	288,958.82	Mississippi
Missouri	28,989,166.92	13,736,014.85	1,543.2	5,336,817.94	2,347,472.10	160.9	15,971,990.70	6,341,521.53	433.3	342,099.68	143,422.14	5.0	218,005.38	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	819,458.01	487,231.02	56.4	1,984,894.66	1,679,352.17	162.4	913,774.29	506,109.37	99.1	4,418,726.55	Montana
Nebraska	11,533,401.62	5,474,202.52	1,768.3	1,000,643.23	493,466.59	103.6	13,828,929.84	6,760,495.70	1,448.3	549,528.26	262,795.00	62.3	1,644,275.19	Nebraska
Nevada	7,558,195.51	5,130,934.59	538.8	1,504,547.76	1,308,267.18	145.1	2,309,406.57	1,983,504.16	266.5	78,768.60	69,095.79	11.5	303,413.28	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6				1,536,842.84	696,021.06	44.4				96,020.87	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	1,321,609.25	264,655.29	15.2	7,898,314.85	2,700,498.56	45.4	1,087,730.42	250,995.00	16.7	152,928.94	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0				2,829,243.51	1,927,724.72	224.7	267,533.70	165,350.34	26.7	1,539,653.56	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	2,054,794.43	740,976.18	45.2	37,845,123.00	10,009,870.20	633.6	7,513,300.00	1,745,447.50	98.2	3,636,943.93	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9	3,621,683.44	1,412,891.77	73.3	5,464,235.08	2,497,344.60	153.8	1,335,706.60	629,631.69	53.7		North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	1,661,628.40	795,770.59	210.9	6,665,365.24	3,530,457.07	936.4	528,473.66	264,236.77	80.0	126,334.79	North Dakota
Ohio	47,689,532.90	17,371,787.03	1,364.1	1,668,165.08	712,351.45	59.8	13,236,973.61	5,137,017.18	400.9	1,719,468.32	514,710.00	32.1	1,995,930.34	Ohio
Oklahoma	28,247,950.33	13,159,999.15	1,178.9	872,674.87	425,274.55	37.0	2,658,104.70	1,107,658.72	100.9	2,396,706.25	956,809.93	145.9	410,044.65	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	702,784.65	403,640.92	23.7	3,018,523.75	1,635,338.95	120.6	232,363.19	139,417.91	7.4	107,734.43	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1,188.8	250,508.96	47,888.62	3.2	29,042,660.25	8,291,474.66	582.7	3,426,073.22	1,076,158.45	68.4	362,527.23	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	274,781.19	113,520.00	7.6	1,245,917.75	326,130.00	21.8	764,687.46	209,880.00	13.9	459,209.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1,481.9	1,046,962.27	398,353.63	54.0	5,840,403.71	2,550,512.35	198.4	99,046.24	42,921.58	10.3	43,413.51	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,181.2	949,031.24	468,642.17	163.6	3,797,788.55	1,925,242.26	657.3	44,142.53	24,278.39		144,800.21	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	947,593.41	448,148.71	34.8	8,683,001.65	3,857,158.99	238.1	1,312,094.83	399,784.45	42.5	298,914.83	Tennessee
Texas	69,183,673.48	27,440,254.72	4,920.2	3,074,600.03	1,333,924.00	226.6	17,082,593.67	7,560,903.80	710.6	4,492,220.89	2,170,333.77	122.2	2,071,014.71	Texas
Utah	8,253,178:03	5,098,440.68	546.4	393,597.51	293,480.96	29.4	1,716,879.71	1,297,322.97	158.9	857,511.03	559,555.59	52.0	569,978.80	Utah
Vermont	4,242,042.64	2,017,699.61	134.5	21,059.66	10,529.83	0.1	2,248,735.41	874,948.98	42.3	41,251.89	13,104.61	0.7	352,224.07	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.5	994,947.57	453,729.25	34.5	5,943,722.43	2,556,461.06	166.7	347,790.04	27,640.23	6.4	77,955.35	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	300,577.74	116,642.49	8.7	3,716,334.38	1,810,600.00	52.9	757,108.08	261,000.00	46.7	174,624.05	Washington
West Virginia	9,473,716.44	4,141,052.65	392.9	457,126.16	193,696.41	12.4	5,534,119.93	2,182,384.47	153.8	1,396,733.60	619,160.80	67.4	216,206.67	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1	101,729.42	50,664.71	9.3	8,836,656.57	4,201,185.88	391.9	352,994.47	169,917.50	13.7	2,624,341.18	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.5	665,939.52	424,150.00	80.3	2,686,023.34	1,707,301.34	205.8				393,935.61	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	53,893,735.77	25,072,881.13	2,812.5	371,039,920.21	156,698,405.74	14,900.1	48,729,409.48	17,314,258.89	1,855.7	46,110,750.66	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 100,781,053.83 Federal aid $ 43,398,999.98 Miles 3833.1

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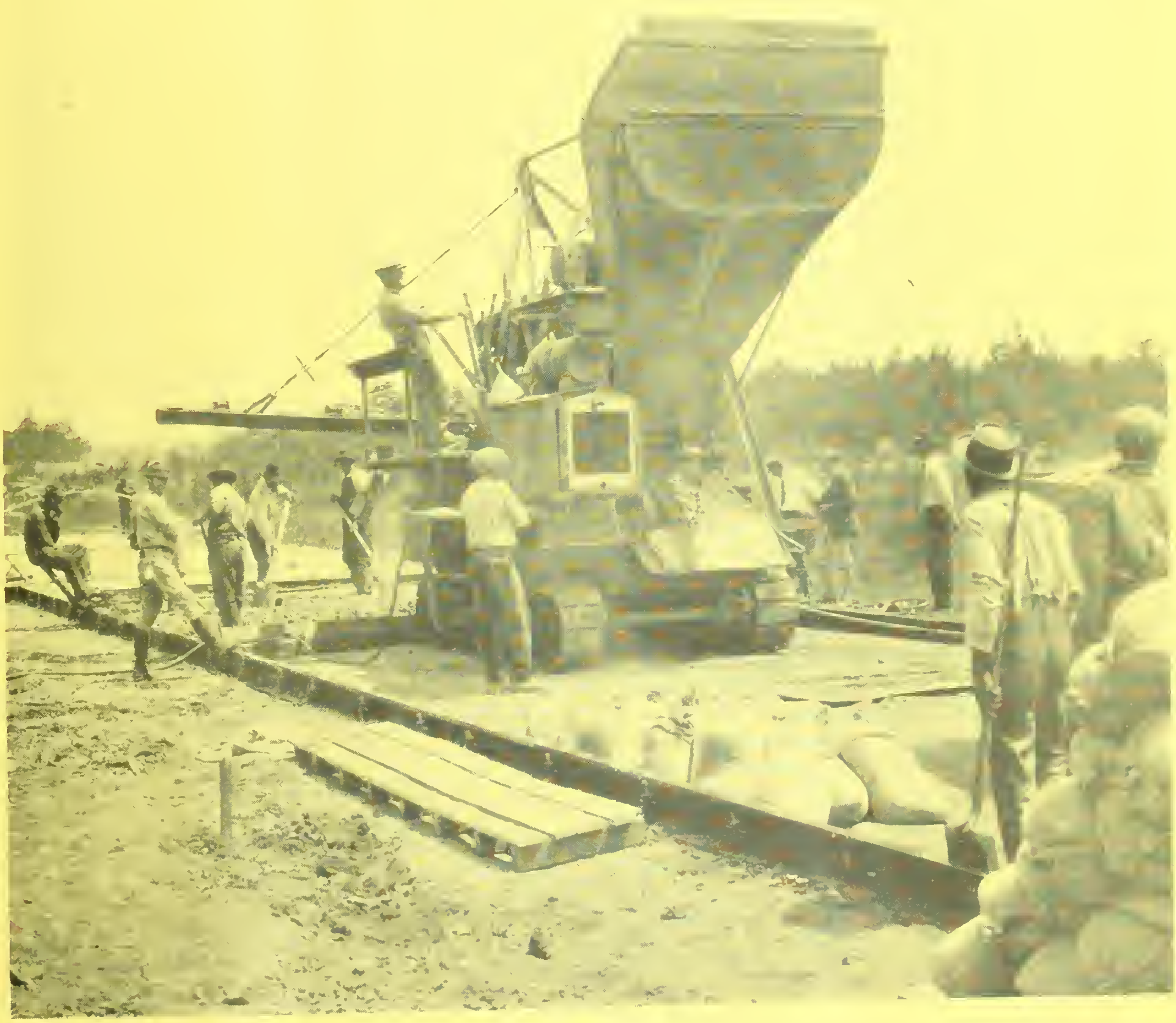

KEEP THE MIXER GOING BY ANTICIPATING PREVENTABLE DELAYS

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TIME LOSSES IN CONCRETE ROAD CONSTRUCTION

Reported by **ANDREW P. ANDERSON**, Highway Engineer, United States Bureau of Public Roads

ALL contractors and engineers familiar with concrete road construction are fully aware that, at best, the actual annual production of any outfit is always far below the potential productive capacity of the mixer. This failure to approach the production possibility of the controlling equipment is due to a large number of reasons which may be grouped under two main classes: (1) Slow or inefficient operation when actually at work; and (2) failure to operate at all for more or less extended intervals during the working season. The more general term of "time losses" is usually applied to this second class.

The first class—slow or inefficient operation—its causes and effects, as well as means and methods for securing more efficient production, has been fully discussed in previous studies reported in this magazine [1] and will therefore be referred to only incidentally in this article.

During the construction seasons of 1925 and 1926 the Bureau of Public Roads in connection with its studies of production engineering made careful records on a large number of active concrete pavement jobs of the time losses and delays which served to hold up production entirely or to reduce it below its normal rate. These studies are still being continued, but the data now cover projects scattered over such a wide region as to form what may be considered a somewhat representative indication of the average conditions under which a substantial portion of our total annual concrete road construction is performed.

If the data accumulated for the past two years are representative of average conditions, of which there would seem to be little doubt, the average concrete paver operating on State road construction does not reach an average of 1,000 hours of actual mixer operation per season, even when supplied with ample work under contract. Some, where contracts are short or conditions otherwise unfavorable, do not exceed 500 hours of actual mixer operation. For the region covered by these studies, which extends from and includes Pennsylvania in the East to Nebraska and Oklahoma in the West, and from Michigan in the North to Texas and Florida in the South, 2,000 hours may be considered as the available working time per season in the more favored sections, except for rather limited areas in the extreme South, and about 1,500 hours as the available time in the less favored areas. This is the length of the average working season under present prevailing practice, exclusive of Sundays and holidays. This report will therefore review the causes which so greatly reduce the actual productive operating time in concrete road construction below that which is apparently available, the effect of such time losses, and what suggestions such data as are now available contain as to how these time losses may be reduced.

INITIAL AND PREPARATORY LOSSES

The first kind of time losses encountered by the concrete road contractor are those involved in securing the contract and in preparation incidental to getting onto the job. These are elusive factors about which sufficient definite data are as yet unavailable. The rather general practice of letting contracts in the spring instead of in the fall or early winter has a tendency to increase these losses. If the contractor fails to secure a contract at the first spring letting a considerable portion of the season may be lost before another letting is held; or where, as is so often the case, when the earthwork and structures are let as separate contracts these contractors may fail to push their work with sufficient vigor to permit paving operations to begin as early in the spring as would otherwise be possible or to carry them on consistently during the season. The actual extent of these losses, however, is largely unknown except as they serve to reduce the length of the average construction season.

The present studies have, in general, covered only the more tangible losses occurring after the arrival of the contractor on the job in the spring until work is discontinued in the fall. Furthermore, in these studies only definite cessations of mixer operation, each of 15 minutes or more in duration, have been classed as time losses. All of the unnumerable stops of less than 15 minutes each have been considered rather as the natural manifestations of slow or inefficient operation than as due to the more or less accidental or fortuitous causes or circumstances to which we are generally accustomed to ascribe time losses. For a full discussion of such minor and accumulative time losses as are due to inefficient management, the reader is referred to the previously mentioned series of articles entitled "Efficiency in Concrete Road Construction."

On this basis it appears from the data accumulated that what might be termed the average concrete road construction outfit from the time it arrives on the job until it is closed down for the winter actually pours concrete during less than 53 per cent of the available working time within this period. Nearly one-half of the time the outfit is on the job and should be engaged in productive work is therefore lost for various reasons. Tables 1 and 2 show the general average distribution of these time losses for the past two working seasons. The first column of Table 1 shows the percentage of the total available working time lost on account of more important causes, while the second shows the percentage which each of these causes contributed to the total time losses. Table 2 shows the percentage of the total available working time lost on each of 32 projects on account of the more frequently occurring causes.

TABLE 1.—*Average time losses by the mixer in concrete paving construction*

Percentage of total working time	Percentage of total time loss	Cause
21.5	45.4	Rain and wet subgrade or wet road.
1.8	10.2	Moving, generally from one set-up to another.
4.3	9.0	Waiting for subgrade to be prepared.
4.1	8.6	Hauling equipment slow or inadequate.
3.5	7.4	All miscellaneous causes.
2.6	5.5	Mixer trouble and repairs to mixer.
3.0	6.3	No materials at yard or source of supply.
1.8	3.8	No water, pipe line and pump trouble.
1.1	2.3	Cold weather.
.5	1.1	Time limit on fills.
.2	.4	Waiting on finishers.
47.4	100.0	

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[1] "Efficiency in Concrete Road Construction," PUBLIC ROADS, vol. 6, Nos. 9, 10, 11, 12; vol. 7, No. 1.

TABLE 2.—*Percentage of total available working time lost on account of various causes on several projects*

Project No.	State	Dates (inclusive)	Total time losses	Weather losses		Moving	Subgrade not prepared	Hauling equipment	Lack of material	Mixer trouble and repairs	Water	Time limit on fills	Finishing	Loading plant	Miscellaneous
				Rain and wet subgrade	Cold weather										
1	Texas	Mar. 24 to June 2	60.3	39.3		3.0	10.5	2.3	1.5	0.6	0.8				2.3
2	Oklahoma	Oct. 27 to Nov. 19	44.7	13.5	10.7	9.4	.8				6.0			2.5	1.8
3	Texas	Mar. 1 to Apr. 30	49.7	45.5			1.3		.9	.6	1.4				
4	Missouri	Apr. 7 to June 6	40.3	28.8		1.0	1.1	5.3		.9					3.2
5	do	May 2 to July 31	43.5	30.0		1.9	5.0	2.2		.3			0.7		3.4
6	do	May 27 to July 31	38.8	23.9		1.8	8.5	.8			1.3		1.0		1.5
7	do	July 10 to July 31	44.7	9.3				27.5		1.4			1.5	1.8	3.2
8	do	Apr. 22 to Sept. 18	29.8	21.6		6.2		.4							1.6
9	do	Aug. 1 to Aug. 25	36.0	25.6			5.8								4.6
10	do	Aug. 18 to Aug. 27	29.4	17.3		9.8				1.4					.9
11	Nebraska	Sept. 14 to Sept. 30	25.0	23.7											1.3
12	Missouri	Aug. 10 to Nov. 19	45.8	25.6	3.9	4.9	1.3			2.1	.9			7.1	
13	do	June 12 to Nov. 24	48.4	29.6	2.6	6.3	2.8	3.0		1.7	.4				2.0
14	do	Oct. 9 to Nov. 24	62.9	36.0			15.4	.8		.5	4.2				6.0
15	Illinois	Apr. 30 to Nov. 7	48.5	21.1	2.5	4.1	2.6	4.5	1.1	5.3	6.0				1.3
16	do	May 18 to Nov. 6	53.6	19.8	.2	12.8	2.0	8.7		5.2	.5				4.4
17	do	May 18 to Nov. 5	46.7	13.3	5.1	12.9	.8	10.4	.5	2.6	1.1				
18	Florida	Oct. 9 to Jan. 19	64.4	12.9	1.1		30.8	2.7			1.1	9.7			6.1
19	do	Apr. 25 to Jan. 28	44.8	17.6		6.3	1.9	1.1	8.6	5.5	1.3		.3		2.2
20	Mississippi	Oct. 24 to Dec. 7	57.9	23.0			11.4		7.7	3.0	3.1		.6	6.0	3.1
21	Texas	Apr. 16 to July 31	51.8	14.9		2.4	4.0	2.9	10.7	10.5	2.5			.5	3.4
22	do	June 15 to July 11	58.0	35.8		5.9	1.9	1.3	9.8	.4	.5				2.4
23	do	June 17 to Aug. 1	41.6	3.5			19.2	3.3		.2	.4			11.0	4.0
24	do	Aug. 9 to Aug. 21	64.1	26.1			24.6			1.3	.6				11.5
25	Missouri	May 24 to Sept. 25	55.8	29.6		7.5	3.6	.3		2.3	9.6		1.0	.1	1.8
26	Michigan	June 7 to Sept. 11	34.0	10.3		1.8	3.2	5.8	1.3	1.7	.1		.3	3.7	5.8
27	do	June 14 to Sept. 11	48.4	15.8			3.6	7.9	13.5	.5	.9			.2	6.0
28	do	June 21 to July 10	26.0	.3			8.0	1.9		.8	3.5		7.3		4.2
29	Missouri	June 14 to Sept. 4	55.0	24.7				20.4	2.4	3.6	1.5		.5		1.9
30	Oklahoma	June 14 to July 17	28.9	20.7			1.6		5.0						1.6
31	do	June 21 to July 31	35.0	16.8		8.9	1.3	.5	1.8	1.4	1.6		.1		2.6
32	do	Aug. 9 to Sept. 11	50.1	16.0		2.1	.4	.3	7.3	.4	6.0	13.1		3.7	.8
	Average		47.4	21.5	1.1	4.8	4.3	4.1	3.0	2.6	1.8	.5	.2	.6	2.9

The most striking item in these tables is the relatively large amount of time lost because of rain and wet subgrade. It is very difficult to determine just how the losses should be divided between actual rain and wet subgrade. Apparently, however, less than one-fifth of the total time losses here charged to the combined causes were due to the actual fall of rain during working hours. On one fairly typical job the total losses charged to rain and wet subgrade amounted to 385¾ hours, and of this total only 53 hours were due to actual rainfall during working hours. The detrimental effect of the rainfall on the subgrade or the road or track over which the hauling is done is therefore clearly the major factor in this class of time losses.

When so large a proportion of the so-called rain losses are caused by the effects of rain rather than by the actual rainfall, it becomes pertinent to inquire whether or not these effects can in some way be mitigated so as to reduce the time which ordinarily must elapse from the cessation of the rainfall until operations can be resumed. Similarly, equipment will fail to function or break down, materials will fail to arrive according to scheduled orders, men will quit, and a hundred and one other things can and do happen to delay or prevent the pouring of concrete. The total of the time losses from all these various causes is even greater than the total weather losses, and therefore these also deserve careful consideration.

PREVENTABLE AND UNPREVENTABLE LOSSES

As a matter of practical consideration, the time losses and delays incident to present methods of concrete-road construction may be divided into those which can be prevented and those which can not. But such prevention as may be possible can only be had by the exercise of considerable thought and attention and at some cost. The problem therefore becomes, in part, one of balancing the probable losses against the probable gains.

Table 3 gives in detail the time losses on a typical job during a period of three weeks. On this project there were a number of time losses, amounting in the aggregate to 43½ hours, which a little foresight and careful attention on the part of the management would have entirely eliminated or greatly reduced. For example, the spare mixer cable could just as readily have been carried with the mixer as left at the main office, and the wait of an hour and a quarter could thus have been eliminated without extra cost. Similarly, when the new cable was installed the clamps could have been properly tightened just as readily as not and 3 hours and 50 minutes of further loss could thus have been prevented. On the other hand, a few planks to bridge the heavy trucks over occasional soft spots in the subgrade would have required some outlay in the first instance and would have entailed some expense for keeping them with the outfit and available as needed, but this expense would have been small in comparison with the value of the 3½ hours lost because of their absence. Table 4 contains a summary of the time losses on this job for a period of four months, classified so as to group together those which were unavoidable on the one hand, and those which could have been avoided by intelligent management on the other.

Perhaps one of the major reasons why so few road contractors make any really systematic effort to reduce their time losses is that they do not know with any certainty the probability of such losses under given conditions. All contractors, of course, know that time losses and delays do occur and that they maker serious inroads into their expected or hoped-for profits. But, in general, such disruptions of their production plans or schedules are usually charged simply to luck or to the impossibility of foreseeing or preventing them.

Yet a number of such disruptions and delays can be shown to have a sufficiently high probability to be worthy of very serious consideration. Table 2, which is a summary of the time losses on 32 projects for more or less extended time periods, shows that while no two jobs suffered exactly to the same extent or in the same way, yet none escaped losses of rather serious proportions. The most common losses were from rain and wet subgrade even on jobs which ran for comparatively short periods during midsummer, but all weather losses together and the moving loss, which is generally unavoidable, account for only a little more than half of the total delay on the average project. Of the other losses, which are largely avoidable, the most serious were those caused by lack of prepared subgrade, by insufficiency or breakdown of the hauling equipment, by lack of materials, and by mixer and water troubles. Of the 32 projects listed in Table 2, 27 suffered to a greater or less degree from lack of prepared subgrade, 23 from insufficiency of hauling equipment, 14 from material shortage, 25 from mixer trouble, and 24 from water trouble.

There are therefore three important questions confronting the contractor: (1) The kind, persistence, and extent of his present time losses; (2) the cost of such losses; and (3) the cost and certainty of the measures which may be adopted to eliminate or reduce the loss.

No two jobs exhibit exactly the same conditions. The data presented in this paper merely illustrate tendencies and possibly indicate very roughly the average probabilities. Each contractor should keep an accurate daily record as to the amount and cause of all losses, and this record should be summarized weekly, monthly, and for the job and season. Such data will soon form a real working basis from which to judge the value of efforts to reduce the losses shown to be occurring on his own jobs. General averages as well as particular data from jobs other than his own, unless all the attendant conditions are known, are apt to prove misleading if they are considered as other than general indications of the possibility of loss.

TIME LOSSES EXPENSIVE

Many contractors fail to realize the actual extent of the losses incurred when production is interrupted. The amount appearing on the pay roll is usually

TABLE 3.—*Time losses on a typical job classified as avoidable and unavoidable*

Date	Time losses				Cause
	Unavoidable		Avoidable		
	Hrs.	*Min.*	*Hrs.*	*Min.*	
July 5			1	00	Lack of subgrade.
			2	15	Finishing machine off of forms; poor forms.
July 6			1	30	Water pump out of order.
				30	Late start.
			3	00	Mixer operator sick; no substitute provided
	1	30			Rain.
July 7	10	30			Wet subgrade.
July 8	3	30			Do.
	5	45			Rain.
July 9	10	30			Wet subgrade.
July 10	6	45			Do.
			1	00	Setting forms.
July 11			1	30	Soft spot in subgrade; no planks available.
			2	00	Roller trouble due to neglected repairs.
July 12				40	Lack of subgrade.
				20	Water trouble.
	3	30			Gasket blown on mixer engine.
July 13				30	Water-pump trouble due to neglected repairs.
			1	00	Resetting forms; faulty engineering.
	1	00			Moving mixer over bridge.
July 14				45	Lack of subgrade.
			1	15	Water trouble.
July 15				45	Do.
			1	15	Mixer cable broken; had to send 4 miles for new cable.
	1	30			Replacing mixer cable.
			1	00	Lack of subgrade.
July 16			1	00	Water trouble.
			4	00	Lack of subgrade.
July 17			3	45	Do.
				30	Finishing-machine trouble; poor forms.
July 19			1	00	Water trouble.
			1	30	Finishing-machine trouble; poor forms.
			1	00	Mixer cable pulled loose; not properly tightened when installed.
July 20			1	30	Mixer cable again pulled loose.
				15	Loader trouble.
				30	Lack of subgrade; fault of inspector.
				30	Water trouble.
				30	Finishing-machine trouble; poor forms.
July 21			1	00	Soft spot in subgrade; no planks available.
			1	00	Lack of trucks.
July 22			1	00	Cleaning water tank.
			1	00	Soft spot in subgrade; no planks available.
				15	Loader trouble.
July 23				30	Truck mired down at stock pile.
				15	Mixer out of gas.
				15	Poor inspection.
July 24			1	20	Mixer cable pulled loose again.
				40	Lack of subgrade.
Total	44	30	43	30	

TABLE 4.—*Analysis of time losses during the working season on one job from May 25 through September 25, 1926* [1]

	Hrs.	Mins.
Total time mixer operated during the construction season	447	50
Time lost in avoidable delays:		
Water trouble: Old water pumps; old 2-inch pipe line and inadequate supply of pipe line	87	30
Preparing the fine grade ahead of and in the rear of the mixer	62	3
Truck shortage	32	35
Improper subgrade drainage	35	30
Mixer trouble	19	0
Making unnecessary move with paving outfit	17	0
Repairing old, worn-out subgrade roller	9	30
Poor engineering and inspection	9	0
Using old forms that would not support the finishing machine	7	55
Getting late start in morning	5	40
Miscellaneous delays	4	12
Outfit stopped work for an expected rain; no rain came	2	45
Setting up new finishing machine	2	30
Using 2 old worn-out finishing machines	1	40
Loaders at loading plant		45
Total avoidable delay	297	35
Total time mixer should have operated during the construction season	745	25
Time lost in unavoidable delays:		
Wet subgrade due to previous rain	297	25
Rain during working hours	53	0
Moving outfit to new location	54	15
Mixer, mechanical trouble	13	10
Miscellaneous delays	1	15
Total unavoidable delay	419	5
Total number of working hours in construction season from May 25 to Sept. 26, 1926 (average length of working day, 10 hours 30 minutes)	1,164	30
	Miles	
Total production during construction season	8.56	

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[1] Delays of less than 15 minutes duration occurring during the hours that the mixer was in operation are not shown in this analysis.

accepted as the measure of these losses, whereas it is far from the true measure. On a job on which the pay roll amounts to only from $200 to $300 a day, the total fixed charges which must be met during the working season may readily amount to from $15,000 to $20,000. Included in these fixed charges, in addition to the straight-time or continuing pay roll, are the interest and depreciation on plant and equipment which, on a modern job involving pavement construction only, may readily exceed $50,000 and even reach $100,000. As improved equipment is constantly being introduced and as the up-to-date contractor feels that he must keep step with such improvements, the amount of the fixed charge for equipment is a product of obsolescence rather than wear and tear, and depreciation is properly chargeable on a straight-time rather than an actual operating-time basis. And the same is true of the other fixed charges, including insurance, taxes, truck licenses, the cost of securing the job, and getting to and from it, and no inconsiderable part of the pay roll—all are practically fixed for the season and independent of the number of hours of actual production.

All these costs and expenses as well as the profits expected must be earned during the hours of actual production; and, as has been stated, the fixed charges may readily amount to from $15,000 to $20,000. Consequently, if concrete can be poured during 1,000 hours the fixed costs will average somewhere between $15 and $20 per hour; and if only 500 hours can be so utilized during the season the hourly charge to pay these fixed costs must be raised to $30 or $40 per hour. Obviously, with fixed charge so heavy little or nothing is left for profits, even though the hourly rate of production be fairly high. Hence it follows that every hour of available time which can be converted from nonproduction into normal production represents a very definite financial gain to the contractor.

Some kinds of time losses are more expensive than others. Thus, interruptions to actual production, such as waiting for trucks or trains to deliver materials, waiting while subgrade is prepared or while a break in the water pipe line is being repaired, usually involve full-time payment of the entire crew; whereas full or half day lay offs because of more prolonged interruptions, such as rain and wet subgrade, failure of materials to arrive at the yard or serious breakdowns of the controlling equipment, usually involve only the full-time payment of a comparatively small portion of the personnel. If, therefore, the lay off is not protracted it may be less expensive than the numerous short-time interruptions which occur during the working day. If, on the contrary, it is prolonged, men are apt to quit and the organization otherwise becomes more or less disrupted and then a serious loss results. Very rarely do we find production during the first day after a protracted period of idleness proceeding at its normal rate. Sometimes the effect is clearly apparent for two or three days.

LOSS CAN BE REDUCED BY CAREFUL MANAGEMENT

To what extent and with what degree of success the various kinds of time losses can be reduced will depend very largely on the ingenuity and resourcefulness exercised by the management. Poor management is certain to produce proportionately large time losses. Conversely, the employment of a really high-grade, able job superintendent is the best insurance against excessive losses from these causes as well as the best and probably the only road to really consistent and efficient production.

Intelligent production managers have long sought to prevent, or at least to reduce, interruptions and delays through a careful planning of the details and sequence of every operation, together with an intelligent forecast of the difficulties which may be encountered and the means to be adopted for their elimination. In such work it is not uncommon to find considerable expenditures devoted entirely to decreasing or preventing the possibility of certain probable delays or interruptions. The justifiable limit of such expenditures is dependent on the cost which such interruptions or delays would incur, the probability of their occurrence, and the degree of certainty with which they can be decreased or eliminated. In other words, expenditures of this kind are wise and profitable up to some point less than the lower limit of their actual insurance value. Beyond that point it is better to accept the loss if it comes than to make the relatively large expenditures necessary for reducing the probability of its occurrence.

To some extent these modern management methods in regard to interruptions and delays to production are in general use in concrete road construction. Thus, cement and often sand and stone or gravel are ordered well in advance of all probable construction requirements as a simple matter of insurance against delays due to the possible failure of such materials to arrive in exact conformity with the construction requirements. The cost of the extra handling, interest charges, stock-pile losses, etc., are often considerable. Yet many contractors find the incurrence of these extra costs a profitable venture. Many contractors also make it a practice to carry in stock on the job a supply of certain repair parts for the controlling equipment, and find that they are able thereby greatly to reduce many delays that otherwise would prove long and costly.

These are some of the lines which concrete road contractors have rather generally adopted in their efforts to insure continuity of production. Whenever intelligently applied in conformity with the requirements of the particular job the results have generally been satisfactory. The data assembled by the Bureau of Public Roads during the past two years clearly show the benefit of such precautions and point to other fields in which modern management methods might possibly find further extension with equally beneficial results.

SUBGRADE DRAINAGE SAVES MONEY

Examples of points at which a beginning for such extension might be made will readily come to mind. Thus, on one job during the past summer 35½ hours of possible mixer operation were lost because of the continued failure to provide means for subgrade drainage. Forty cents worth of common labor each day of the hundred or more in the working season would have made ample provision for any water which might fall during the night to escape to the side ditches. But no provision was made, with the result that during each rain water collected between the forms as in a pond until such time as men could be sent out to dig the necessary outlets. During one 10-day period this occurred three times. Here, then, is a case where about $50 worth of common labor plus a small amount of direction and forethought would have saved the contractor during the season at least 35 hours for additional production—worth in this particular case at least $1,200. This one item may also offer some explanation as to why this contractor out of a total of 1,164½ available hours from the opening of the season until September 26 succeeded in utilizing only 448 hours in actual production. Of the total time losses, 386 were charged to rain and wet subgrade, although rain actually fell only during 53 hours of the working time. Further details of this project are given in Table 4.

It is in meeting and overcoming conditions of this kind that intelligent, resourceful management will find its greatest field of economic usefulness. Better methods for providing surface drainage as well as for handling the actual operations of subgrading and form setting with a view to mitigating the effects of rainfall are undoubtedly possible. Such methods might, in the first instance, be considerably more expensive than the prevailing methods and yet prove economical providing they can be shown to be dependable in materially reducing the time losses below the normal expectancy under present methods.

Frequently the roads over which the materials for new construction must be hauled are unimproved or in a very bad state of repair. Roads of this kind require a considerable amount of well-directed, systematic maintenance to make possible such orderly operation of the hauling equipment as will eliminate mixer delays without the use of a large oversupply of trucks. Such maintenance is also usually essential to the early resumption of hauling after rains. Yet, in spite of these well-known prerequisities and the further well-known fact that such maintenance will often more than pay for itself in lower hauling costs and less wear and tear to the equipment, one rarely finds a contractor who makes even a pretense of systematic maintenance. In the few cases which came under the observation of bureau representatives, where maintenance was attempted it was generally assigned to some one entirely without experience in the work. Possibly it is the fact that this is generally the case that is responsible for the idea, firmly established among road contractors, that the maintenance of unimproved roads will not pay. The probability that it will pay if it is properly handled is indicated by the fact, that, on the typical projects observed by the bureau representatives during the past two years, an average of 8.6 per cent of the total time losses was caused by the hauling equipment. This does not include the time losses due to the road being too wet or soft for hauling, although the subgrade was dry enough to permit the pouring of concrete. The total losses suffered under this condition are not known. As the distinction is rather difficult to make, such losses are generally charged to rain and wet subgrade. However, there was one job on which the distinction was made; and on that job, which showed about an average distribution of time losses, more than 10 per cent of the total losses charged to rain and wet subgrade were really due to the fact that the hauling road was too wet or soft for operation of the hauling equipment. On this job the contractor lost about $150 per month because the road did not dry sufficiently fast to permit hauling as soon as the subgrade was dry enough for pouring concrete. Careful, systematic maintenance would, no doubt, largely have eliminated this loss.

OTHER PREVENTABLE LOSSES

Moving from one set-up to another during the working season seems to consume much more time than is generally believed. The average for all the jobs studied during the past two years was 10.1 per cent of the total time losses, the second largest item on the list. Sometimes more moves are made than conditions really warrant. The economic factors involved in determining the number of set-ups which should be made on a given job are fully discussed in previous issues of this magazine.[2] The amount of time consumed in making these moves, however, seems to be so large that a considerable reduction should be possible when the contractor fully understands the actual, monetary loss incurred every day his equipment is nonproductive. In selecting equipment one of the factors which should be given careful consideration is that of mobility or ease of dismantling, transporting, and reassembling, not only of individual units but also for the entire plant. The various details of the entire operation of moving should be planned as carefully as the operations involved in the placing of the concrete.

Waiting for the subgrade to be prepared accounted for 9 per cent of the total time losses on the projects studied. These losses should be possible of entire elimination. With very few exceptions they are wholly chargeable to poor management. On the average job, as shown by these studies, their entire elimination would be worth from $2,000 to $3,000 per season—a sum sufficient to more than pay the difference in salary as between a poor and a good superintendent.

The remaining time losses, such as those due to poorly operated or insufficient hauling equipment, mixer trouble, lack of materials at the supply yard, water supply, and the various miscellaneous causes, all of which comprise about 33 per cent of the total average time losses or about 16 per cent of the total available time, are also very largely a question of management. Few, if any, of these losses can be entirely eliminated without incurring excessive expenditures. But there would seem to be little reason to doubt that all can be reduced and some very materially. This is shown so clearly on the observed jobs on which the management was of an especially high order as to be subject to little or no question.

Thus, on one job blessed with an exceptionally able superintendent but otherwise fairly typical of the many jobs studied, the total time losses from all causes during the period from April 22 to September 18, inclusive, consumed less than 30 per cent of the total available working time as compared to 47 per cent for the average job. (See Table 2, project 8.) As the season was rather rainy and the soil a heavy clay, the losses due to rain and wet subgrade were about average, or 21.6 per cent. Moving from one set-up to another, or from one section to another, also required about the normal amount of time; but all other time losses were reduced to a remarkably low percentage, and some were eliminated entirely. Thus no time was lost waiting for subgrade to be prepared nor from faulty operation of the mixer, nor because of the water supply. In fact, the total time losses on this job during the period indicated from all causes other than rain, wet subgrade,

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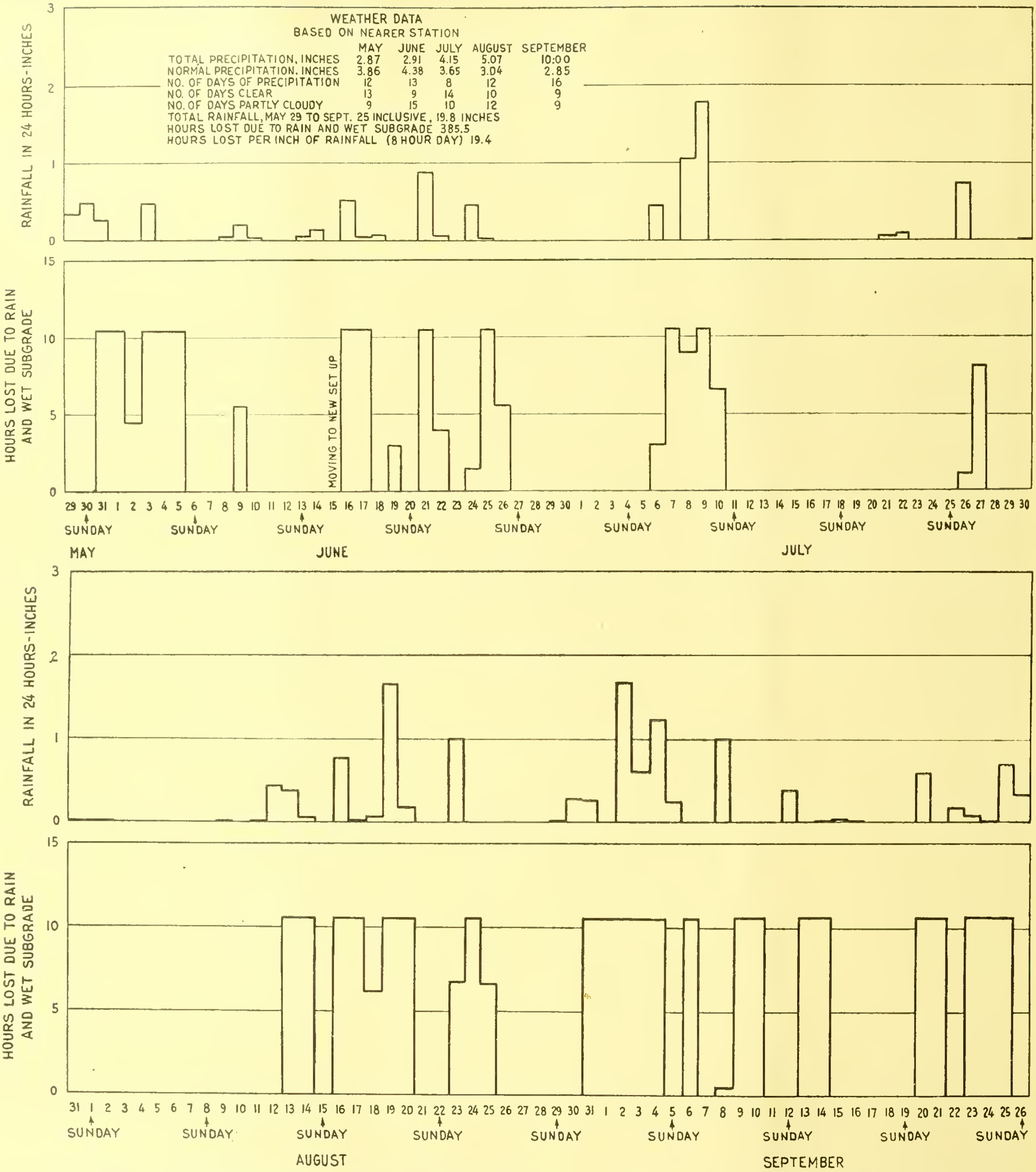
[2] "Efficiency in Concrete Road Construction," PUBLIC ROADS, vol. 6, Nos. 9, 10, 11, 12; vol. 7, No. 1.



WEATHER DATA
BASED ON NEARER STATION

	MAY	JUNE	JULY	AUGUST	SEPTEMBER
TOTAL PRECIPITATION, INCHES	2.87	2.91	4.15	5.07	10:00
NORMAL PRECIPITATION, INCHES	3.86	4.38	3.65	3.04	2.85
NO. OF DAYS OF PRECIPITATION	12	13	8	12	16
NO. OF DAYS CLEAR	13	9	14	10	9
NO. OF DAYS PARTLY CLOUDY	9	15	10	12	9

TOTAL RAINFALL, MAY 29 TO SEPT. 25 INCLUSIVE, 19.8 INCHES
HOURS LOST DUE TO RAIN AND WET SUBGRADE 385.5
HOURS LOST PER INCH OF RAINFALL (8 HOUR DAY) 19.4

Fig. 1.—Rainfall and Time Losses from Rain and Wet Subgrade on Job Near Columbia, Missouri, from May 29 to September 25, 1926

and moving amounted to only 2 per cent of the total potentially available working time as compared with 20 per cent of the available working time lost from these causes on the average job during the season.

Even in the short working season, of 1,500 available hours, this decrease of the nonproductive time from 300 hours for the average job to only 28½ hours on this particular job represented an actual money value of more than $8,000 if we consider only the direct loss the firm would have sustained if the outfit had remained idle the longer period. But on this job the rate of production was exceptionally high, not just for short periods but was so maintained consistently throughout the entire season. Thus ever hour of available working time utilized in actual production doubtless represented a considerable profit. While this project

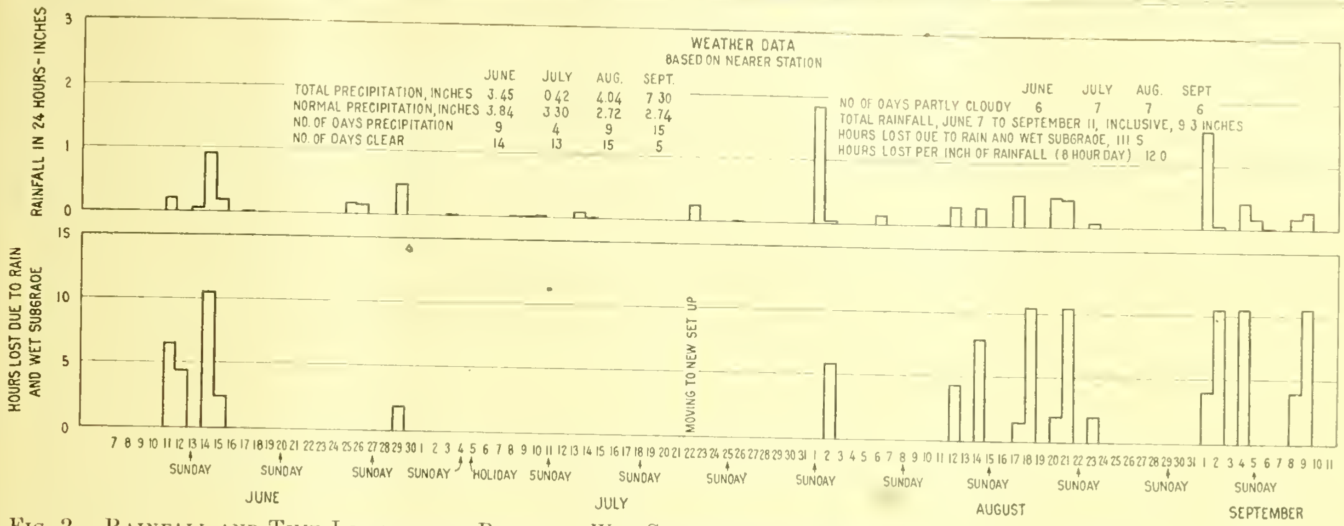


FIG. 2.—RAINFALL AND TIME LOSSES FROM RAIN AND WET SUBGRADE ON JOB IN BOWIE COUNTY, TEXAS, FROM JUNE 7 TO SEPTEMBER 11, 1926

TABLE 5.—*Time losses, in hours, in summer and fall*

Project No.	Dates (inclusive)	Available working hours		Lost time									
		Mixer in operation	Mixer idle	Rain, wet subgrade, etc.	Moving	Subgrade not prepared	Hauling equipment	Lack of materials	Mixer trouble or repair	Water	Cold weather	Finishing	Miscellaneous
13	July 1 to 31	212.5	74.5	46.0			2.0		16.0			2.0	8.5
	Oct. 1 to 31	76.0	215.0	171.0			1.0		3.0		40.0		
19	July 1 to 31	188.5	72.0	28.0		10.75	2.75	21.75	4.75	53.0		1.0	
	Oct. 1 to 31	117.5	146.0	39.0	38.0	1.0	22.25	28.0	9.5	2.0		0.75	5.5
15	July 1 to 31	186.0	99.0	12.0	53.0	13.0	10.0		2.0	8.0			1.0
	Oct. 1 to 31	134.0	265.5	150.0	32.0				36.5		47.0		
16	July 1 to 31	155.0	142.0	22.0	69.0		29.5		21.5				
	Oct. 1 to 31	110.5	206.5	127.0	20.5	9.0	13.0		8.0		3.0		26.0
17	July 1 to 31	165.75	104.25		66.75		25.75		7.75	4.0			
	Oct. 1 to 31	118.0	169.0	79.0	10.0		17.0				63.0		
Grand total		1,463.75	1,493.75	674.0	289.25	33.75	123.25	49.75	109.0	17.0	153.0	3.75	41.0
Total for July		907.75	491.75	108.0	188.75	23.75	70.0	21.75	52.0	15.0		3.0	9.5
Total for October		556.0	1,002.0	566.0	100.5	10.0	53.25	28.0	57.0	2.0	153.0	0.75	31.5

offers the most outstanding example of the benefits of good management so far encountered, it differs from several others only in degree. All the studies indicate that the relative size of the preventable time loss recorded is a fairly good indication of the quality of the management on the job. Poor management seems to be invariably associated with relatively large preventable or reducible time losses, while under able management these time losses are reduced to relatively minor importance.

WHAT OF THE WEATHER LOSSES?

Rainfall and its effects on the subgrade and the road or track over which the materials are hauled, together with frost and cold weather, was the cause of nearly one-half of the average time lost on all jobs studied. These losses naturally vary greatly from job to job. A sandy or gravelly, porous soil will dry far more quickly than a heavy clay or silt. The amount and distribution of rainfall also differs from place to place. No constant relation between the amount of rainfall and the time lost because of rain and wet subgrade is therefore to be expected. However, it may be possible under given conditions to approximate the probable loss of time by consultation of past rainfall records, and such a forecast is valuable as an estimating guide even if nothing can be done to reduce the time loss.

To ascertain what relation existed on the projects observed a number of them were selected for special study with this point in mind. No attempt was made to measure the rainfall actually occurring on the job; but data were obtained from the nearest weather station for the period of the job. In some cases the stations were located very close to the job. But, as rainfall often varies considerably in a short distance, there can be no definite assurance that the records represent the exact rainfall as it occurred on the road. Nor is assurance necessary. The contractor or superintendent who wishes to learn something of the weather that may be expected on a prospective job in order that he may make a suitable allowance for delays and be prepared to meet and overcome the resulting difficulties generally will have no more definite source of information than the past records of the nearest weather recording station. As kept by most stations, these data show the rainfall, temperature, and state of weather during each 24 hours, and the normal or average condition for each month—all of which may be of importance in forecasting the probable conditions on any given job.

Figure 1 shows the weather and rainfall during the past season on one road that was largely on new location. The soil beneath the surface was fairly heavy clay on the hills and heavy silt or gumbo along the streams. About the only indicated relation is that a half inch or more of rainfall caused a delay of at least one entire working day. Smaller rainfall, if limited to one day, apparently caused relatively large or small time losses depending on the time of day when it

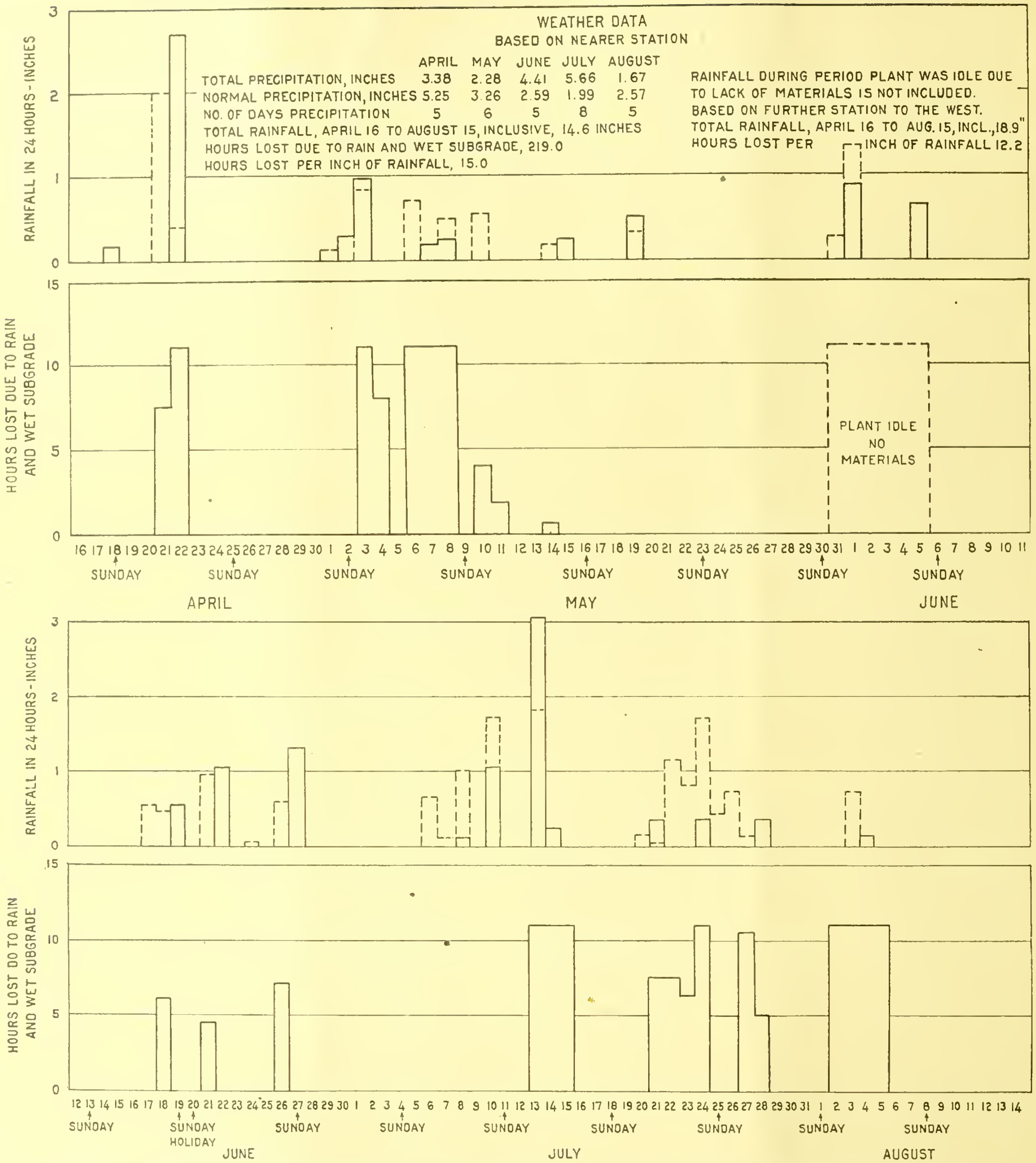
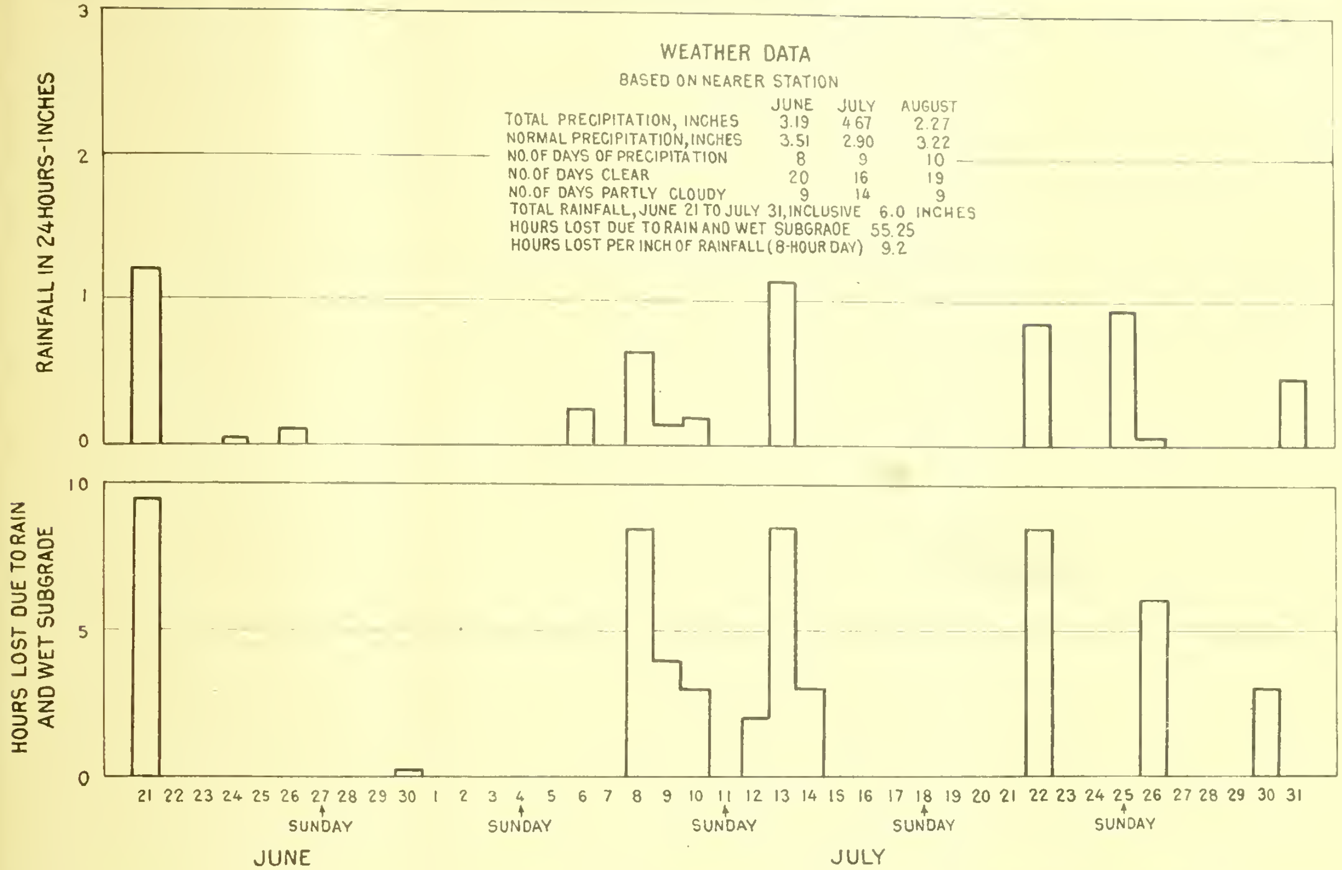


Fig. 3.—Rainfall and Time Losses from Rain and Wet Subgrade on Job in Bowie County, Texas, from April 16 to August 15. Rainfall at Nearest Station to the East Shown by Full Lines and that at the more Distant Station to the West Shown by Dash Lines

occurred. For the period May 29 to September 25, inclusive, we find that an average of 19.4 hours working time were lost for each inch of rainfall.

Figure 2 shows another job located on an old right of way and a somewhat lighter soil. Here, apparently, a rainfall of nearly ¾ of an inch was necessary to cause a delay of a full working day. Here also, for the period involved, the average time lost per inch of rainfall was only 12 hours.

Figure 3 shows the rainfall as recorded at two stations between which another project was located. It will be noted that the rainfall was considerably different at these two stations and that the time losses on the project do not correspond very closely to what would be expected from the rainfall measured at either station. The last time loss of three full days was largely due in the first part to a local rain which did not extend to either station. Based on the rainfall as



WEATHER DATA

BASED ON NEARER STATION

	JUNE	JULY	AUGUST
TOTAL PRECIPITATION, INCHES	3.19	4 67	2.27
NORMAL PRECIPITATION, INCHES	3.51	2.90	3.22
NO. OF DAYS OF PRECIPITATION	8	9	10
NO. OF DAYS CLEAR	20	16	19
NO. OF DAYS PARTLY CLOUDY	9	14	9

TOTAL RAINFALL, JUNE 21 TO JULY 31, INCLUSIVE 6.0 INCHES
HOURS LOST DUE TO RAIN AND WET SUBGRADE 55.25
HOURS LOST PER INCH OF RAINFALL (8-HOUR DAY) 9.2

Fig. 4.—Rainfall and Time Losses from Rain and Wet Subgrade on Job Near Guthrie, Oklahoma, June 21 to July 31, 1926

recorded at the nearest weather station, and eliminating the time loss due to the local rain, we find that each inch of rainfall during the period involved caused an average loss of 15.0 hours of working time while if we use the rainfall at the other station as a basis only 12.2 hours of working time were lost per inch of rainfall. Figure 4, illustrates further the relation between rainfall and the working time lost to actual production.

FALL WEATHER LOSSES LARGE

There are so many variable factors which enter into the relation between rainfall and time losses that the data so far available from the present studies warrant no definite conclusions except in regard to one point; namely, that an equal amount of rainfall and equal distribution under a given condition will cause a much longer delay in the fall or early spring than in the summer. Consequently, the contractor who permits possible hours of summer operating time to escape in the hope of making up lost time by operating a little later in the fall or starting operations a little earlier the next spring is gambling against heavy odds. Table 5 gives the time losses on a number of fairly typical jobs for the months of July and October, respectively. Here we find that in spite of numerous attempts to work overtime on good days in October the time losses averaged 64 per cent in October as against 35 per cent in July for these jobs. This difference is almost entirely due to the time losses caused by rain, cold weather, and wet subgrade.

The time of day at which rainfall occurs, its intensity or duration, as well as the time between rainfalls probably under some conditions affect the amount of time lost more than the actual amount of the rainfall. Comparatively small rainfall occurring during working hours will stop work and if falling on a subgrade but little past the point permitting work will generally cause a much longer delay than the same amount falling under identical conditions except on a thoroughly dry subgrade.

Although the data herein presented are not sufficient to warrant definite conclusions, they do indicate that certain very similar conditions exist on many projects, and that these conditions have a tremendous influence on the actual cost of the completed work. They indicate also that nearly half of the average time losses are due to causes which are or can be made subject to the control of the contractor. With increasing skill and ability in management we may expect to find this type of time loss reduced in the future. The other half of the losses due directly and indirectly to adverse weather conditions it is clearly impossible to eliminate entirely and such losses are therefore amenable only in part to managerial control. All of the data so far assembled, however, seem to indicate that a goodly proportion of the time lost now as the direct or indirect result of rainfall is not inherent in the nature of the physical conditions, but rather, at least in part, is due to difficulties inherent in or connected with the present methods of operation and management. Therefore as the probability ratio of the various forms of loss and the financial burden they place upon the contractor become better known, the faults of our present means and methods will surely be corrected.

If, under certain conditions, we find that on the average $50 worth of attention to surface drainage of the prepared subgrade is reasonably certain to save for

(Continued on page 208)

TESTS OF CONCRETE CURING METHODS

A PROGRESS REPORT OF EXPERIMENTS CONDUCTED BY THE BUREAU OF PUBLIC ROADS

Reported by **J. T. PAULS**, Highway Engineer, United States Bureau of Public Roads

FOR THE purpose of obtaining information as to the effect of moisture and temperature changes in concrete road slabs immediately after laying, and to determine the effect of various curing methods and subgrade friction on the concrete, the Bureau of Public Roads built, during August, 1926, at Arlington, Va., a series of 40 concrete slabs, each 200 feet long by 24 inches wide and 6 inches thick. The concrete was of 1 : 2 : 4 mix with Potomac River sand and Potomac River gravel as aggregates. The mixing and placing were done under weather conditions as nearly identical as possible. A view of the slabs in place is shown in Figure 1.

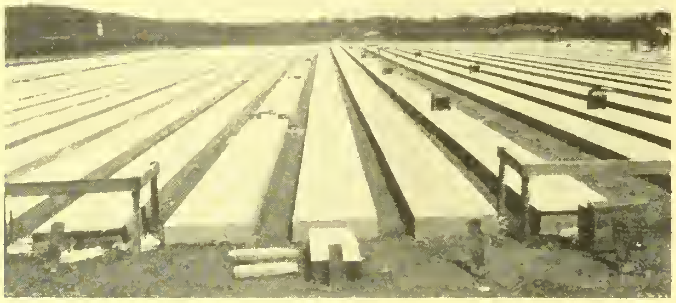


FIG. 1.—GENERAL VIEW OF THE EXPERIMENTAL SLABS AT ARLINGTON, VA.

The plan of construction and curing, as detailed in Table 1, covered a wide range of variables so that, after sufficient time and study, information should be available which will permit a better understanding of the following subjects:

1. Comparative effect of different methods of curing.
2. Effect of steel reinforcing of various types and weights in distributing stresses resulting from shrinkage in the concrete.
3. The effect of expansion joints in distributing stresses resulting from shrinkage in the concrete.
4. The effect of moisture in the subgrade in reducing shrinkage cracking.
5. The effect and prevention of rapid drying of the newly placed concrete.
6. The magnitude and effect of subgrade resistance.
7. The curling of concrete pavements.
8. The possible relation between pavement behavior and test data from control specimens.

TABLE 1.—*Details of construction and curing of the experimental slabs*

Slab No.	Subgrade condition	Reinforcing	Length of segmental sections (feet)	Curing method
1	Dry	None	None	None.
2	do	do	20, 30, 40, 50, 60	Do.
3	do	do	None	Tarvia B.
4	do	do	do	2 pounds calcium chloride.
5	do	do	do	Sodium silicate.
6	do	do	do	3 pounds calcium chloride.
7	Dry	None	None	Tarvia B; tar paper on subgrade.
8	do	2 ¼-inch deformed bars.	do	None.
9	do	2 ⅜-inch deformed bars.	do	Do.
10	do	2 ½-inch deformed bars.	do	Do.
11	do	2 ¾-inch deformed bars.	do	Do.
12	do	2 ¾-inch plain bars painted and greased.	do	Do.
13	do	2 ½-inch deformed bars.	20, 30, 40, 50, 60	Do.
14	do	2 ¾-inch deformed bars.	do	Do.
15	do	None	None	2 per cent calcium chloride admixture.
17	do	43.8-pound mesh	do	None
18	do	23.6-pound mesh	do	Do.
19	do	43.8-pound mesh	20, 30, 40, 50, 60	Do.
20	do	None	None	Dry straw.
21	do	do	do	Dry earth.
22	Lightly sprinkled.	do	do	Wet earth.
23	do	do	20, 30, 40, 50, 60	Do.
24	do	do	None	Wet straw.
25	Wet	do	do	Burlap and wet earth.
26	do	do	20, 30, 40, 50, 60	Do.
27	do	do	None	Burlap and 2 pounds calcium chloride.
28	do	do	do	Burlap and sodium silicate.
29	do	do	do	2 pounds calcium chloride.
30	do	do	do	Sodium silicate.
31	do	do	do	2 per cent calcium chloride admixture.
32	do	43.8-pound mesh	do	Burlap and wet earth.
33	do	23.6-pound mesh	do	Do.
34	do	43.8-pound mesh	20, 30, 40, 50, 60	Do.
35	do	2 ¼-inch deformed bars.	None	Do.
36	do	2 ⅜-inch deformed bars.	do	Do.
37	do	2 ½-inch deformed bars.	do	Do.
38	do	2 ¾-inch. deformed bars.	do	Do.
39	do	2 ½-inch deformed bars.	20, 30, 40, 50, 60	Do.
40	do	2 ¾-inch deformed bars.	do	Do.
41	do	None	None	Asphalt emulsion.

Sufficient time has not elapsed since the investigation was started to make it possible at this time to give complete results. However, on a few of the problems studied sufficient indications have been obtained to warrant presentation at this time. These somewhat scattering data and preliminary indications are presented in the following paragraphs.

LOSS IN MOISTURE

Two methods were used to obtain the loss in moisture of the concrete. The first consisted of taking a pan of concrete (about 30 pounds) and weighing it as it came from the mixer; afterwards placing it in the sun and weighing at hourly intervals. In this manner the loss of moisture from the surface of the concrete, without curing, was determined.

The other method consisted of pouring slabs 24 by 12 by 6 inches concurrently with each long slab, and weighing the concrete as it came from the mixer. The slab was then cured in the same manner as the corresponding long slab, the edges of all the small slabs being painted after 24 hours with a heavy tar. In this manner the loss in moisture was limited to the surface and the base. The slabs were weighed 24 hours after pouring and at intervals thereafter. Figure 2 gives the moisture loss obtained under the several different conditions.

Curve A in Figure 2 represents the loss in weight of a slab laid on a dry subgrade and without any curing treatment. It can be seen from this curve that the greatest loss in weight and hence in moisture content occurred during the first 24 hours after pouring. Curve B represents the loss in weight of a similar slab laid on a wet subgrade, covered with wet burlap for 24 hours and wet earth for 13 days. This curve also shows that the greatest loss in moisture occurred during the first 24 hours after pouring. The loss in moisture in the slab laid on the dry subgrade without curing is approximately four and one-half times the loss that occurred in the slab laid on the wet subgrade and cured with wet burlap during the first 24 hours after pouring. The lower curve in Figure 2 represents the loss in moisture over a period of 28 hours, using 30 pounds of concrete, with evaporation limited to the surface. The greatest loss by this method was found to occur during the first three or four hours after the concrete was poured.

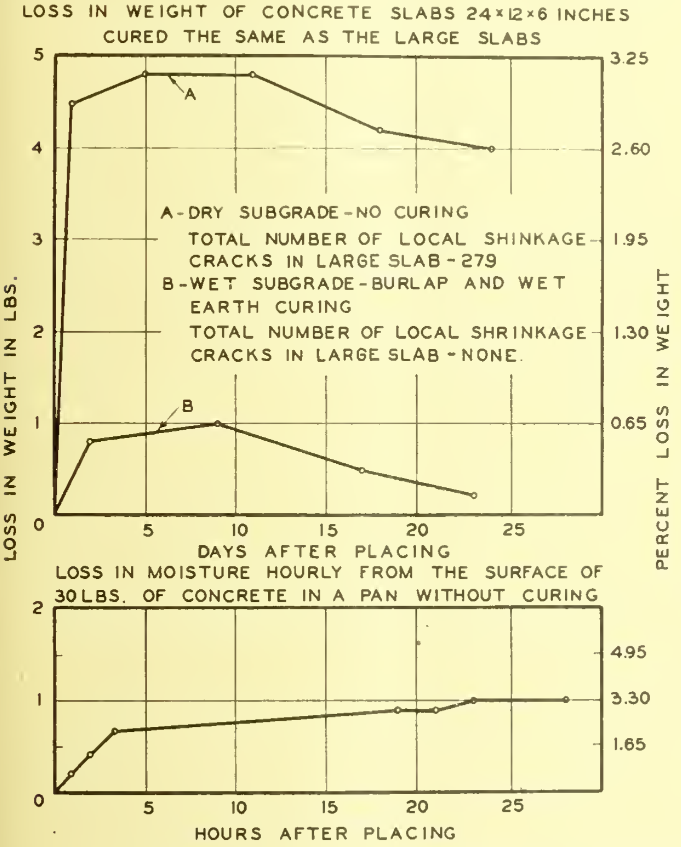


Fig. 2.—Diagrams Showing Loss of Moisture in Concrete

The effect on the concrete of the rapid drying out during the first few hours after placing is shown by the number of local shrinkage cracks occurring. In the slab with no curing 279 local shrinkage cracks developed, while none occurred in the slab which was covered with wet burlap during the first 24 hours.

It seems definitely indicated that the most important requisite of a satisfactory curing process is that it shall prevent or retard the drying of the concrete during the first full 24 hours after placing.

EFFECT OF STEEL REINFORCING

A series of reinforced slabs was included as one of the features of the investigation, the purpose being to determine the effect of the different sizes and types of steel on the distribution of stresses developed in the pavement under different conditions of curing from causes other than traffic.

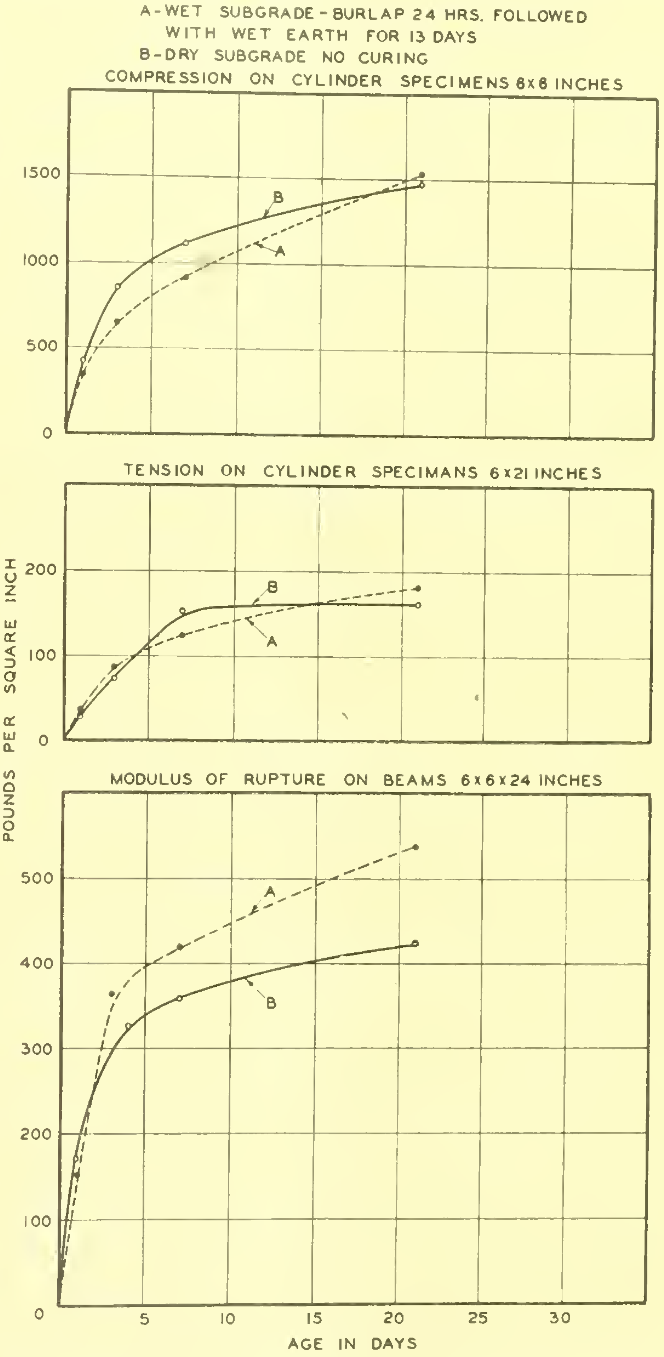


Fig. 3.—The Effect of Curing on the Strength of Control Specimens

Three types of reinforcing were used in the different slabs. They were: Deformed bars of several sizes placed continuously and segmentally; plain ¾-inch bars painted and greased; and rectangular mesh of 23.6 and 43.8 pound weights. All reinforcing was placed 3 inches below the surface of the pavement. In the case of the bar type, two rods were placed in each slab 12 inches apart and 6 inches from the edges.

Table 2 shows the coefficients of shrinkage obtained with several different types and quantities of reinforcing in comparison with the coefficient of unreinforced concrete. The shrinkage coefficient was determined from microscopic measurements of the width of the transverse cracks in the different slabs.

Table 2.—*The effect of reinforcing in reducing the thermal coefficient of expansion and contraction*

Slab No.	Reinforcing	Area of steel in the slab section	Number of transverse cracks	Average total width of transverse cracks	Averago coefficient (per degree C.)
		Square inch		*Inch*	*Inches per inch*
1	None	None.	6	0.532	0.000011
8	2 ¼-inch deformed bars	0.130	4	.288	.0000066
9	2 ⅜-inch deformed bars	.20	4	.199	.0000045
10	2 ½-inch deformed bars	.386	15	.125	.0000024
11	2 ¾-inch deformed bars	.868	6	.023	.00000064
12	2 ¾-inch plain bars painted and greased	.910	6	.470	.0000095
17	43.8-pound mesh	.240	5	.108	.0000025
18	23.6-pound mesh	.132	3	.261	.0000061

These measurements show that certain types and amounts of reinforcing continuously placed greatly reduce the thermal coefficient of the concrete pavement.

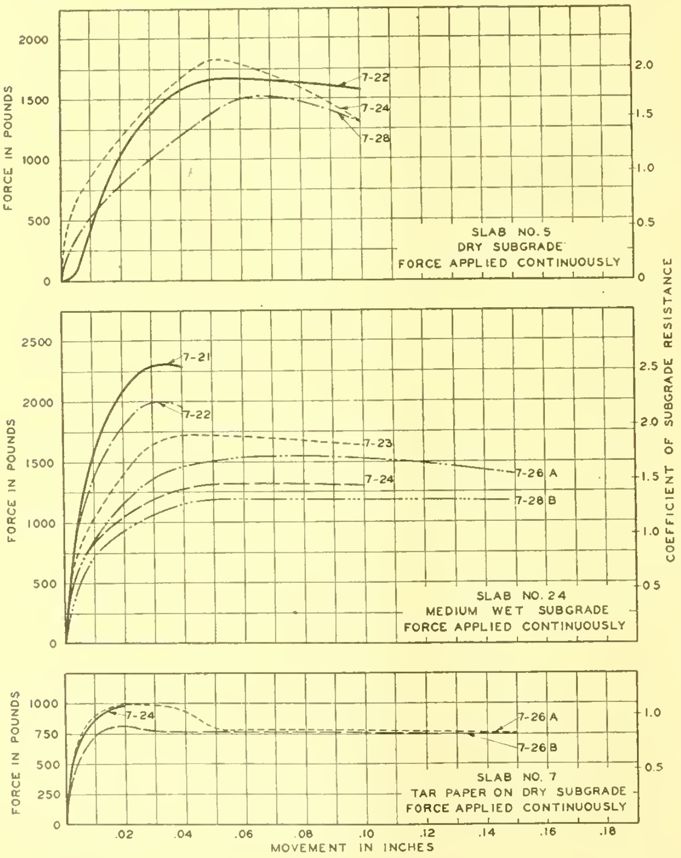


Fig. 4.—Diagrams Showing Resistance to Movement of Slabs by Dry, Wet, and Tar-Paper-Covered Subgrades

The slabs reinforced with two continuous ¼-inch deformed bars and with rectangular mesh of 23.6-pound weight are found to have approximately the same coefficient, and it will be noted that the cross-sectional area of the steel in each case was nearly the same.

The greatest reduction in the shrinkage coefficient was obtained by the use of two ¾-inch deformed bars, and the minimum reduction—almost negligible in amount—was obtained with the use of two ¾-inch plain bars painted and greased.

TESTS OF CONTROL SPECIMENS

Compression, tension, and modulus of rupture specimens were made concurrently with and cured similarly to the large slabs. Specimens for compression were 6 by 6 inch cylinders; for tension, 6 by 21 inch cylinders; and for modulus of rupture, 6 by 6 by 24 inch beams. Two sets of each were made to be tested at 24 hours, 3, 7 and 21 days and 6 months.

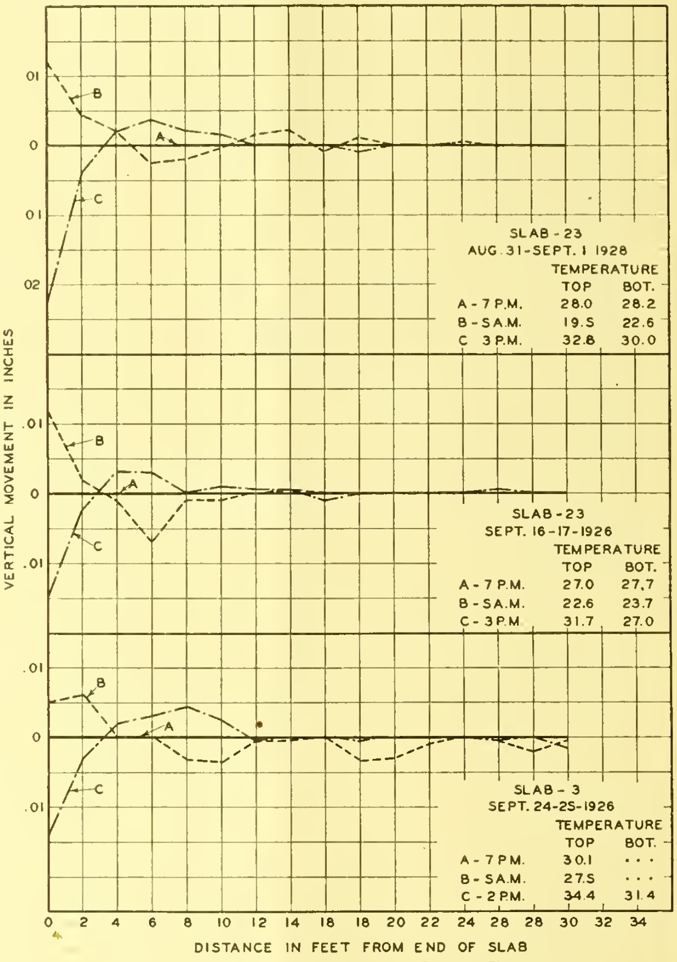


Fig. 5.—Measurements of Slab Curling

All of these, except the six-month specimens, have been tested at the present time, and the average strength curves for each type of test are shown in Figure 3. These results show that specimens laid on the dry subgrade without any curing reached a higher early strength than those laid on a wet subgrade and covered with wet burlap for 24 hours and wet earth for 13 days. As the specimens age, those made on the wet subgrade show a higher strength than those on the dry subgrade without curing.

SUBGRADE RESISTANCE

The resistance of the subgrade to the horizontal movement of the slab has been determined for several subgrade conditions. The value of this coefficient was obtained by measuring the force required to move

(Continued on p. 208)

RESEARCHES ON BITUMINOUS PAVING MIXTURES

Reported by W. J. EMMONS, Highway Research Specialist, United States Bureau of Public Roads

DURING the past year research on bituminous mixtures has been vigorously prosecuted by a number of organizations. Although attention at present is focused principally upon the development of a test which will define the resistance to displacement of any mixture when subjected to conditions of service, the basic motive behind all such research is the formulation of a rational theory of design or the substantiation of an existing theory.

At the Bureau of Public Roads two pieces of apparatus are in process of development which it is hoped will assist in the solution of the problem. Neither apparatus is perfected but it is felt that a description of the work being done will prove of interest at this time.

DISPLACEMENT DETERMINATOR

A machine which attempts to duplicate to a certain degree the action of traffic on a pavement surface has been designed for the purpose of determining the comparative strength or resistance to displacement of bituminous mixtures. Figure 1 shows in diagrammatic form the arrangement of this machine. The essential feature is a series of 11 steel cylinders or rolls, 4 inches in diameter by 3 inches long, mounted between and near the peripheries of two confining steel disks, which in turn are rotated by a motor. Beneath the rolls is a water-tight bath or tank in which is placed the specimen to be tested. At the beginning of the test the rolls are lowered gently to the surface of the specimen and the motor started. Rotation of the rolls takes place as they pass over the specimen, tending to deform it longitudinally. A certain amount of impact is also imposed as each roll leaves the specimen and the following one comes in contact with it. A small metal plate held lightly against the end of the specimen and connected with an Ames dial by a brass rod constitutes the device for measuring deformation.

The specimens are prepared by hand mixing and are compacted in a rectangular, collapsible steel mold by means of an electric hammer fitted with a square tamping end. Specimens 8 by 6 by 2¼ inches in size have been used in most of the work thus far, although at present the behavior of a smaller size of specimen, 8 by 4 by 2¼ inches, is being investigated. In each case the face of greater area is exposed to the action of the machine.

As might be expected, widely different test values may be obtained by varying the conditions of the test. An arrangement of the machine and of the specimen was sought which would give a wide range in strength

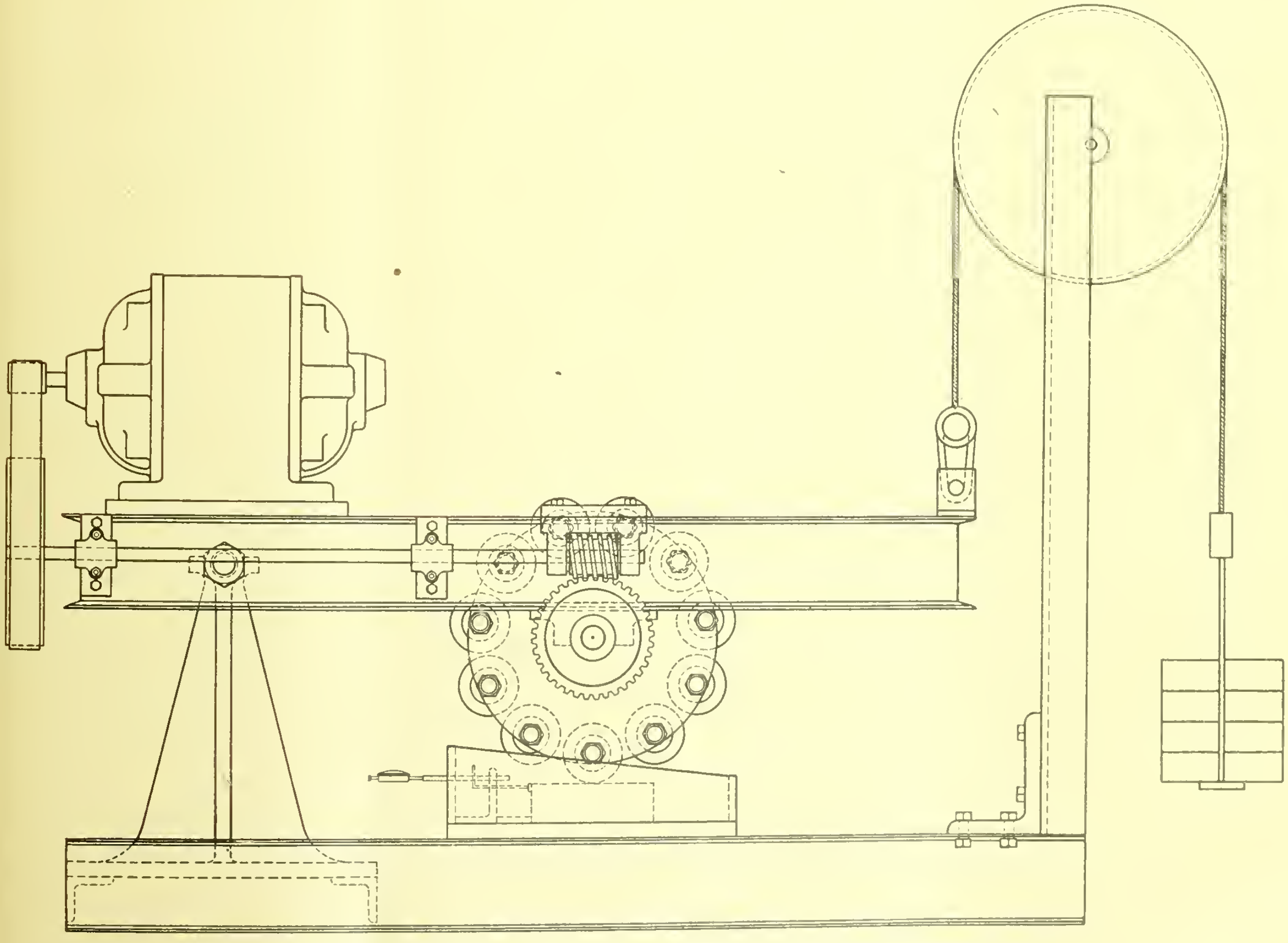


FIG. 1.—ROLLER MACHINE FOR TESTING BITUMINOUS PAVEMENT SPECIMENS

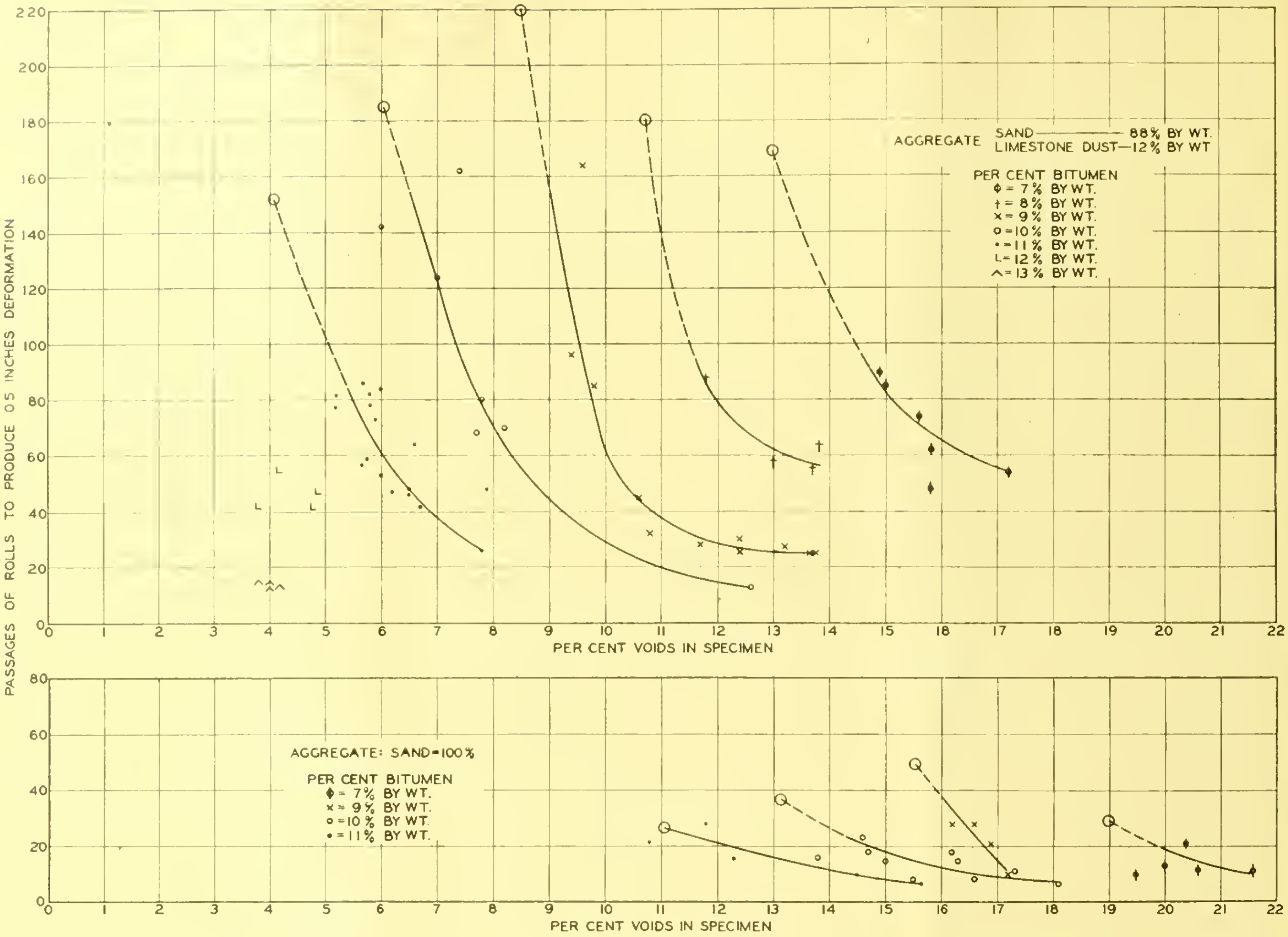


FIG. 2.—RELATION BETWEEN STRENGTH AND COMPOSITION OF EXPERIMENTAL SHEET ASPHALT MIXTURES

values between weak and strong mixtures. The weight imposed by the rolls is susceptible to adjustment by means of counterweights to a maximum of 450 pounds, and the speed of rotation may be varied from 4 to 10 revolutions per minute. The degree of support provided for the specimen under test greatly affects the results; and it has been found best to confine it in a frame at the rear and the two sides. The end toward which the movement takes place has at times been left entirely unsupported but it is probably better to insure againt slipping of the entire specimen by providing small plates partially closing the fourth side of the rectangle.

For the purpose of bringing out the effect of these many variables, rather than of deriving definite information regarding mixtures, many short series of tests have been run. The typical series shown in Figure 2 were made on 8 by 6 by 2¼ inch specimens, with a machine speed of seven revolutions per minute, a load of 250 pounds, and a testing temperature of 60° C. The same aggregate was used in all the specimens represented by each group of curves, the bitumen being varied as indicated.

Several indications of the test are evident from the chart (fig. 2). The test appears to differentiate clearly between mixtures varying in bitumen and dust content. It is very sensitive to slight variations in the density of well-compacted specimens. It is also apparent that a series rather than a single test is required to define the characteristics of any mixture. The failure of certain specimens to check with the average of their respective groups may be due to lack of uniformity in their densities or to certain other conditions of testing or molding which are as yet not clearly understood.

Considerable thought has been given to the method of interpreting curves of this nature. It is evident that data in this form are of little practical value without a knowledge of the degree of density to which mixtures may be compressed in service. Comparatively little information is available, but a study is being attempted which it is hoped will shed light on the matter. As a step in that direction, dry-aggregate voids tests are being made on the aggregates extracted from samples of pavements of different ages, and the results of these tests are compared with the computed voids of the aggregate as it exists in the original sample. Insufficient work has been done to warrant definite conclusions, but from the tests which have been made it is indicated that the voids existing in an aggregate may afford a measure of the compressibility of the aggregate when combined with bitumen.

As a tentative method of comparison between mixtures, the curves of Figure 2 are extended to the point of maximum possible compression indicated by the voids tests upon the aggregates. Unfortunately, these series do not include enough tests to define the slope of some of these curves as well as might be desired, but it seems that with percentages of bitumen which are less than sufficient to fill the voids in the aggregate, a compression can be attained which is, as a rule, within 1 per cent of the computed maximum possible density.

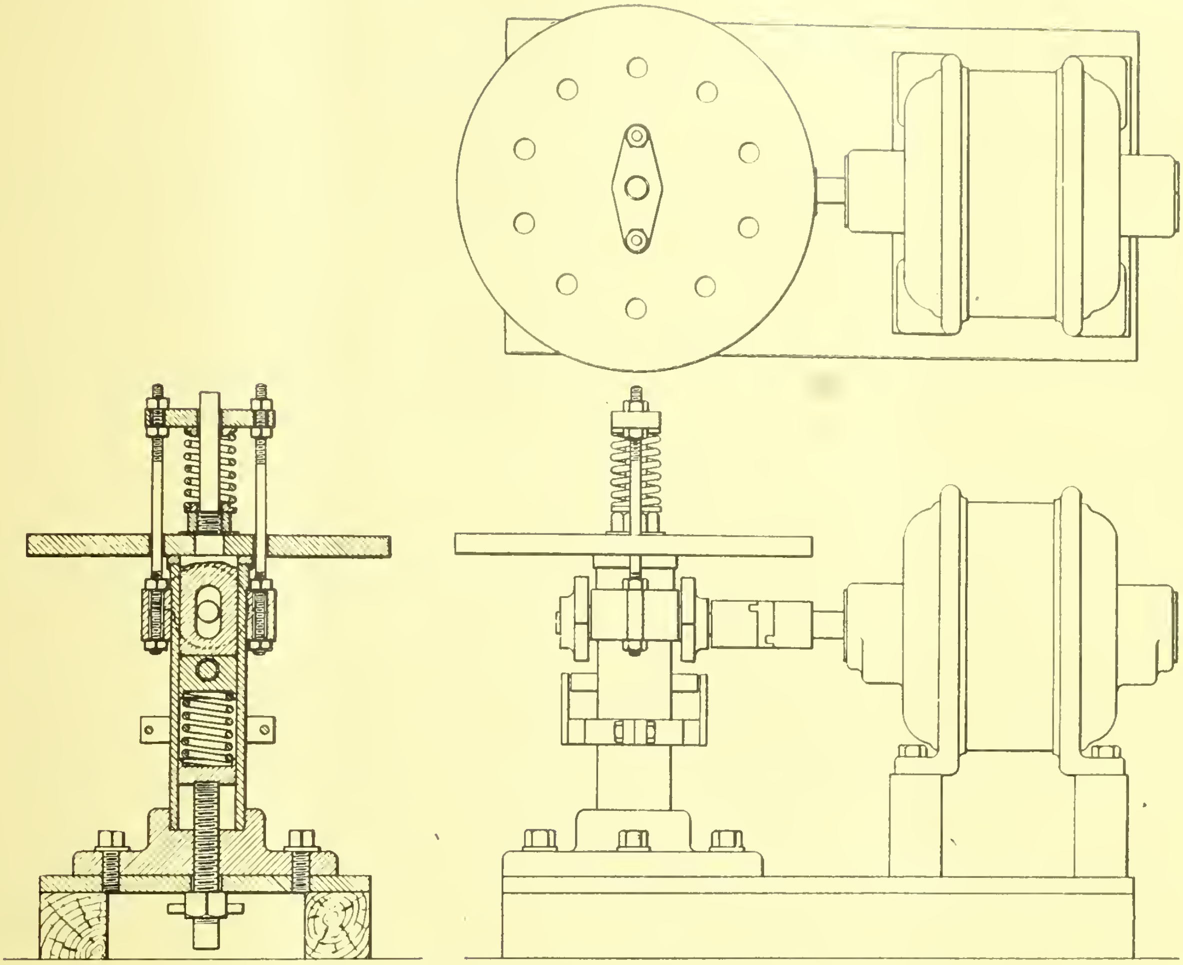


Fig. 3.—Apparatus Used in the Determination of Voids

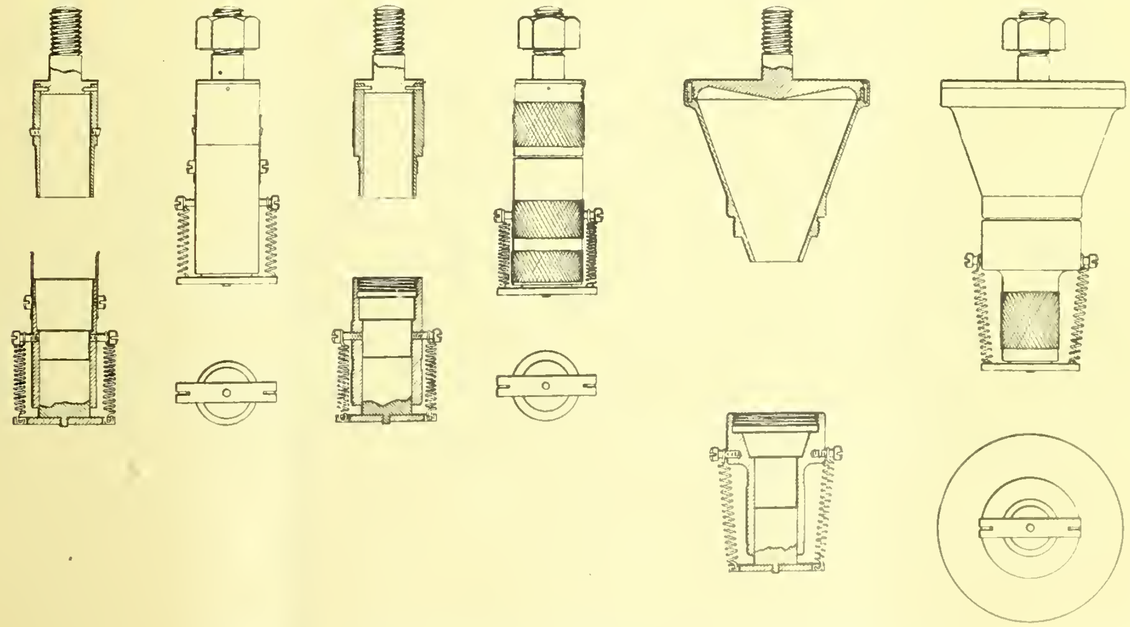


Fig. 4.—Containers Used in Apparatus for Void Determination

VOID DETERMINATOR

Another device developed by the Bureau of Public Roads is a mechanical means for compacting fine-aggregate mixtures in the test for voids. This apparatus is shown in Figure 3. A steel disk, 11 inches in diameter and $\frac{11}{16}$ inch thick, carries the aggregate containers and is vibrated rapidly between the lower and upper springs. Two cams driven by the motor are attached to the shaft, passing through the central column of the machine, alternately compressing and releasing the lower spring at the rate of about 1,500 times per minute. The throw of the disk is adjustable up to a maximum of 0.04 inch.

Figure 4 illustrates the types of containers which have been used to hold the aggregate being tested. The disk of the vibrator is perforated with ten ⅝-inch holes equally spaced on a circle concentric with its circumference. The bases of the containers are equipped with threaded rods by means of which they are bolted to the machine. Either cylindrical or conical containers may be used; and each container is fitted with a removable sleeve which may be attached to it by spring clips or by threads. At the beginning of the voids test, the sleeve is attached, slightly more aggregate than is necessary to fill the calibrated container is introduced, a rubber-shod cylindrical metal plunger is placed over the aggregate, and the whole apparatus is bolted to the vibrator. A 20-minute period of vibration has thus far been employed although it is likely that a somewhat shorter time may be sufficient to produce thorough compaction.

Considerable trouble has been experienced in obtaining a design for the containers which would resist the severe use to which they are subjected. The screw-thread type has virtually been discarded since it was found all but impossible to prevent dust from seeping into and ruining the threads of the containers. The spring-clip type seems to be more durable but as the holes in the clips wear larger it has been found necessary to take up the looseness which develops by wrapping and compressing a rubber band between the shoulder of the container and the sleeve.

Cylinders should be made by boring a solid steel rod in order to insure the greatest rigidity. Certain of the cylinders originally made have lately given erratic results and this has been traced to a very slight looseness which developed between the bases and the walls which, in this case, were turned out separately and assembled.

Most of the work has been done with cylinders 1 inch in diameter and of approximately 26 cubic centimeter capacity. Determinations by the method of hand tamping have been made concurrently with the machine test, using containers of identical construction for both purposes. Voids have also been calculated as they exist in 2-inch diameter cylindrical specimens of sheet asphalt mixtures compressed by the method devised by Hubbard and Field. The aggregates of these specimens were combined with percentages of bitumen from 7 to 14 per cent.

On the basis of the work done thus far it is believed that in the very near future the vibration method may be developed to give at least as complete compaction as can be obtained by either the more laborious hand method or by the method of direct compression under a predetermined load.

(Continued from page 201)

productive operation time worth 20 times that expenditure, then the needed attention to surface drainage will be readily forthcoming. Similarly, if proper attention to the maintenance of the road over which the hauling is done can be shown to yield certain profits above its cost, then such maintenance will become a regular part of the hauling plan. On one point, the data obtained are probably conclusive and that is that under anything like identical conditions the time lost will vary inversely with the ability of the management.

(Continued from p. 204)

slabs 6 feet long and 2 feet wide and 6 inches thick which were cast on the subgrade at the end of a number of the long pavement sections.

A few of the results obtained are given in Figure 4. The resistance is seen to be larger on a medium wet than on a dry subgrade. Tar paper is seen to have greatly reduced the resistance. A large reduction was found to occur also when the slab was moved short distances at 10-minute intervals instead of continuously over longer distances. Partial return of the slab after release of the force is found to take place.

The subgrade material on which these tests were made was a silty loam with very little clay, an exceptionally good material. In view of the favorable conditions under which these high results were obtained, it would seem that the generally accepted subgrade resistance coefficient of 2 might be too small for the average condition.

CURLING OF THE CONCRETE SLAB

The amount of curling of several different slabs has been measured over 24-hour periods. The method employed was to drive stakes along the slab at 2-foot intervals, and on these to mount dials which measured the vertical motion of the slab in thousandths of an inch. In Figure 5 the maximum curling is plotted for several periods of 24 hours. From these curves it can be seen that the maximum movement occurs at the end of the slab; that the maximum bending moment occurs from 6 to 8 feet from the end, and that the curling of the slab is restrained as the distance from the end is increased.

The curling curves shown indicate that a fiber stress of at least 100 pounds per square inch is possible; and it is evident that secondary transverse cracking may occur from this cause before the concrete has attained high strength.

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1926.

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220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
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*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
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*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
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331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

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*62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects. 5c.

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*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying Out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

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Vol. 5, No. 19, D- 3. Relation between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

NOVEMBER 30, 1926

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 19,226,411.34	$ 8,725,985.09	1298.3	$ 1,334,428.79	$ 639,665.69	100.5	$ 5,802,525.54	$ 2,744,634.98	270.0	$ 141,817.96	$ 70,908.93	13.4	$ 2,168,260.31	Alabama
Arizona	10,949,978.25	5,963,772.35	729.8	272,263.37	172,004.74	26.7	1,589,950.50	1,097,546.15	85.0				2,493,925.76	Arizona
Arkansas	18,364,544.50	7,666,698.35	1323.0	925,556.13	452,766.52	74.3	4,213,979.04	1,991,172.27	272.9	1,490,889.48	741,335.08	94.3	753,831.78	Arkansas
California	27,142,696.90	13,003,592.30	1058.0	2,962,908.98	1,471,835.96	105.7	10,427,954.84	5,131,922.61	245.2	335,905.59	127,564.21	7.1	2,337,900.12	California
Colorado	13,905,904.64	7,127,288.18	745.0	886,679.90	492,409.14	32.7	5,602,303.77	2,716,073.46	258.8	538,721.78	299,416.03	30.2	1,700,625.19	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1	415,786.78	153,999.52	8.0	5,753,293.84	1,585,525.34	73.0	244,272.96	86,791.17	3.1	406,979.07	Connecticut
Delaware	4,918,052.29	1,781,665.60	124.3	741,382.64	306,931.14	17.6	915,210.90	373,034.05	25.4				12,427.21	Delaware
Florida	3,832,680.26	1,924,362.32	132.9	2,111,145.68	1,049,833.66	62.5	8,874,502.91	4,028,953.64	227.3	174,536.76	28,095.00	1.8	1,153,709.39	Florida
Georgia	24,791,206.97	11,664,237.86	1794.0	2,416,050.95	1,183,209.73	165.5	11,267,713.99	5,531,487.59	496.3	31,696.14	15,948.05	7.6	37,169.77	Georgia
Idaho	11,061,198.14	5,982,112.70	724.7	1,318,271.26	719,535.29	62.2	2,378,116.51	1,473,377.82	173.0	468,569.16	272,439.27	39.3	212,161.92	Idaho
Illinois	44,116,611.86	20,619,995.74	1377.7	2,012,307.94	980,960.56	68.1	9,556,175.20	4,639,573.48	345.7	1,004,521.91	432,782.21	32.5	3,158,985.91	Illinois
Indiana	16,949,425.87	8,172,125.19	534.3	2,408,698.94	1,135,591.51	71.0	17,470,671.26	8,223,978.57	502.5				672,759.53	Indiana
Iowa	29,062,376.40	11,926,302.10	2114.8	2,969,964.22	1,367,487.09	200.7	12,116,920.08	5,373,707.20	632.6	1,890,781.44	647,339.35	74.7	270,729.26	Iowa
Kansas	32,826,601.64	12,590,439.25	1160.6	522,940.32	235,898.35	48.9	14,211,641.66	5,580,237.15	781.6	2,533,013.33	838,968.54	113.0	218,917.71	Kansas
Kentucky	20,737,706.10	8,492,082.26	758.3	847,024.03	297,523.46	13.3	7,545,749.09	3,586,433.98	383.2	1,326,319.76	656,264.63	62.6	180,504.68	Kentucky
Louisiana	13,830,592.58	6,144,739.99	1055.9	871,444.84	387,478.39	31.9	3,606,456.02	1,734,604.66	185.1	1,392,376.53	640,969.83	42.1	364,715.03	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6	547,274.10	228,564.49	19.1	2,580,964.90	912,095.53	70.7	1,084,926.14	402,821.22	33.2	728,839.37	Maine
Maryland	10,924,943.10	5,112,991.22	423.3	171,244.62	85,622.31	14.9	1,584,527.56	716,072.28	73.6				10,371.18	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.5	412,120.06	121,949.75	5.1	4,965,520.35	1,319,576.75	68.6	920,117.54	232,778.17	14.2	1,777,160.71	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	390,946.77	188,717.04	13.6	14,446,328.78	6,557,259.62	413.2	626,157.00	274,496.50	15.6	1,494,839.54	Michigan
Minnesota	37,170,985.95	15,586,116.56	3181.9	4,812,207.41	2,099,429.11	365.2	5,646,437.58	1,823,600.00	270.5	2,319,448.39	41,000.00	108.6	41,634.33	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1129.0	1,131,550.92	563,149.80	71.7	7,596,137.22	3,698,750.24	377.7	317,835.24	158,917.61	34.9	292,667.26	Mississippi
Missouri	28,989,166.92	13,736,014.95	1543.2	6,125,841.98	2,678,728.20	188.6	15,245,401.71	6,060,545.87	408.6	342,099.68	143,422.14	5.0	167,724.94	Missouri
Montana	11,400,983.81	6,333,465.99	1054.9	858,208.82	509,109.72	59.1	2,039,801.08	1,732,168.83	172.2	817,555.73	429,967.89	87.3	4,420,172.67	Montana
Nebraska	11,533,401.62	5,474,202.52	1768.3	1,669,952.27	823,757.27	183.0	13,281,983.37	6,486,758.49	1392.0	521,763.45	248,964.84	55.5	1,601,551.98	Nebraska
Nevada	7,558,195.51	6,130,834.69	538.9	1,309,262.91	1,575,015.07	183.2	2,065,653.81	1,764,770.48	239.9				324,494.96	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6				1,553,131.16	705,246.06	45.0				86,795.97	New Hampshire
New Jersey	16,346,301.01	5,099,342.21	290.3	1,574,934.83	308,980.29	18.1	7,931,185.48	2,726,328.56	47.2	1,092,069.63	223,605.00	14.9	110,163.94	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1427.0	12,941.78	8,115.76	0.1	2,935,184.15	1,994,922.97	249.9	302,965.70	185,582.18	9.9	1,444,107.81	New Mexico
New York	43,224,279.79	17,911,957.19	1197.0	2,501,509.66	934,544.47	58.1	37,307,973.00	9,908,282.70	620.8	8,070,300.00	1,965,647.50	103.3	3,524,763.14	New York
North Carolina	27,009,419.47	11,177,337.94	1257.9	4,952,692.24	1,923,238.17	102.8	4,786,044.21	2,206,892.72	143.6	876,813.00	408,442.59	34.4	1,294.58	North Carolina
North Dakota	12,313,311.40	6,031,959.78	2183.1	1,933,749.14	881,287.93	222.4	6,480,799.44	3,438,174.18	938.4	513,883.76	256,941.82	79.1	140,395.29	North Dakota
Ohio	47,699,532.90	17,371,787.03	1364.1	2,541,449.91	1,064,321.47	94.1	13,100,141.20	4,972,202.19	381.4	1,399,899.92	561,020.00	17.9	1,762,465.31	Ohio
Oklahoma	28,247,950.33	13,159,999.15	1178.9	900,571.53	439,056.35	41.7	2,650,852.43	1,104,032.68	102.0	2,397,066.86	956,990.24	146.6	399,708.68	Oklahoma
Oregon	17,027,978.42	8,583,214.79	939.2	702,784.65	403,640.92	23.7	3,250,886.94	1,774,756.86	128.0	293,329.42	6,000.00	7.0	101,734.43	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1188.9	250,508.96	47,888.62	3.2	29,958,176.82	8,526,177.50	597.2	2,659,544.04	917,194.42	59.0	286,788.42	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	274,781.19	113,520.00	7.6	1,245,917.76	326,130.00	21.8	917,282.21	250,155.00	16.6	418,834.94	Rhode Island
South Carolina	16,020,639.90	6,766,322.83	1481.9	1,158,643.57	454,194.28	58.0	5,712,030.83	2,491,292.36	194.5	99,046.24	42,921.58	10.3	46,792.86	South Carolina
South Dakota	17,468,373.18	8,603,926.97	2181.2	1,045,179.92	517,229.44	177.8	3,706,472.30	1,978,341.39	643.0	113,029.83	30,539.05	13.0	136,854.15	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	1,189,774.63	567,186.82	44.4	8,603,458.25	3,817,387.29	236.0	1,153,562.01	320,618.04	35.0	298,914.83	Tennessee
Texas	69,183,673.48	27,440,254.72	4920.2	3,690,006.31	1,660,238.48	247.3	17,061,436.34	7,572,699.69	721.1	3,893,068.19	1,884,190.83	126.9	2,049,047.28	Texas
Utah	8,253,178.03	5,098,440.68	546.4	393,597.51	293,480.96	29.4	1,794,506.95	1,354,767.12	166.0	789,883.79	502,111.44	44.9	569,978.80	Utah
Vermont	4,242,042.64	2,017,698.51	134.5	81,136.58	38,579.83	2.0	2,218,938.48	860,003.59	41.1				352,224.07	Vermont
Virginia	21,990,249.44	10,385,728.11	1005.5	1,208,600.77	538,360.38	39.8	5,924,449.90	2,539,832.71	165.9	530,067.63	28,640.23	11.1	8,952.56	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	300,577.74	116,642.49	8.7	4,382,234.09	2,045,600.00	94.9	91,208.37	26,000.00	4.6	174,624.05	Washington
West Virginia	9,473,716.44	4,141,062.65	392.9	457,126.16	193,696.41	12.4	6,125,251.18	2,446,968.32	178.7	1,394,581.91	474,962.60	78.9	95,821.02	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1592.1	101,729.42	50,664.71	9.3	8,890,219.19	4,243,708.88	395.2	120,766.17	60,382.00	4.6	2,691,353.68	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1133.5	739,672.78	471,486.00	85.3	2,613,563.47	1,656,727.22	200.8	8,655.90	5,557.00		391,616.73	Wyoming
Hawaii				343,664.15	97,440.00	6.5	708,620.71	215,195.18	9.4				787,517.82	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	67,111,096.57	31,034,865.48	3,485.8	369,417,694.68	155,589,430.91	14,766.4	45,231,339.33	15,698,391.38	1,694.0	42,373,603.63	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 117,332,109.99 Federal aid $ 50,879,199.26 Miles 4288.1

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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Apparatus Used for Iowa Pipe Tests

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THE COLLECTION AND DISPOSITION OF MOTOR VEHICLE REVENUES[1]

By **HENRY R. TRUMBOWER**, Economist, Bureau of Public Roads

DURING the calendar year 1925 the gross receipts derived from motor-vehicle license fees and gasoline taxes in the United States amounted to $406,648,561. These figures refer only to the revenues collected and received by the several States and by the District of Columbia; they do not include any of the collections made by cities and municipalities on account of local license fees or gasoline taxes. The gross receipts resulting from motor-vehicle registration fees, licenses, permits, fines, etc., amounted to $260,619,621; the total tax earnings on gasoline and motor-vehicle fuel, after deducting all refunds, amounted to $146,028,940. The registration fees, according to these figures, constituted 64 per cent of the total motor-vehicle revenues collected by the States, and the gasoline taxes 36 per cent. The relation of the license fees and the gasoline taxes to the total motor-vehicle revenues for each of the years since 1920 is set forth in Table 1.

TABLE 1.—*Total gross receipts from license fees and gasoline taxes, 1920 to 1925*

Year	License fees		Gasoline taxes		Total motor-vehicle revenues
	Amount	Per cent	Amount	Per cent	
1920	$102,546,212	99	$1,475,136	1	$104,021,348
1921	122,478,654	96	5,302,259	4	127,780,913
1922	152,047,823	93	11,923,442	7	163,971,265
1923	188,970,992	84	36,813,939	16	225,784,931
1924	225,492,252	74	79,734,490	26	305,226,742
1925	260,619,621	64	146,028,940	36	406,648,561

In 1925 the total motor-vehicle revenues were practically four times as great as in 1920; and in the same period the number of registered motor vehicles in the country slightly more than doubled. In 1920 the total motor-vehicle revenues, including license fees and gasoline taxes, averaged $11.27 per vehicle; in 1925 the average had risen to $20.27 per vehicle. The increase in total motor-vehicle revenues was, therefore, due to two causes—an increase in the number of motor vehicles from which license fees were collected, and an increase in the average receipts per motor vehicle on account of the rapidly extended application of the gasoline tax and increases in the fees charged for licenses. The license fees in 1920 averaged $11.18 per vehicle; in 1925 the average was $13.06. The gasoline taxes in 1925 amounted to more than the total license fees collected in 1921 and almost as much as the 1922 license fees. Constituting, in 1925, 36 per cent of the total motor-vehicle receipts whereas in 1920 they were but 1 per cent, it is evident that the gasoline tax revenues have become a substantial part of the country's total motor-vehicle revenues.

These motor-vehicle revenues collected by the States, when compared with the country's total annual expenditures on rural highways, make a very impressive showing. Our rural highway expenditures, according to the best estimate that can be made, amounted in 1925 to $1,288,939,707. It follows, therefore, that the amount collected in the form of license fees and gasoline taxes was equivalent to 31.5 per cent of the total rural highway expenditure. This ratio has been increasing from year to year.

In this study no attempt is made to investigate and analyze the license fee schedules and the gasoline tax rates of the several States which, through their application, have produced these motor-vehicle revenues. Such analyses have been presented in other reports in considerable detail. At the present time it is believed that those concerned with the subject of highway finance will be interested in a study of the disposition and distribution of the annual motor-vehicle revenues by the several States and certain administrative policies and procedures relating to their collection. This study is limited, more or less, to these phases of the subject and is divided into two parts; the first relating to the motor-vehicle license fees, and the second to the gasoline tax revenues.

PART 1. RECEIPTS FROM MOTOR-VEHICLE REGISTRATION FEES, LICENSES, PERMITS, FINES, ETC.

The figures presented herewith cover the total funds received by State and county officials in connection with the operation of the motor-vehicle license laws. The total receipts are recorded by all the States but there are still a considerable number which do not report the detailed sources of receipts. It is hoped that in the future reports more complete details can be presented.

Officers in charge of registration and license fee collection.—In each of the States a State officer or department is made responsible for the collection of the license fees and supervises the enforcement of the motor-vehicle registration laws. In a majority of the States this officer is the secretary of state. There are, however, a number in which the legislatures have created separate motor-vehicle departments and there are others in which the collecting and registering function is delegated to the respective State highway departments. At the present time there are 22 States in which the secretary of state is the responsible officer. These are Arizona, Colorado, Delaware, Georgia, Illinois, Indiana, Iowa, Kansas, Louisiana, Maine, Michigan, Minnesota, Missouri, Nevada, Ohio, Oregon, South Dakota, Utah, Vermont, Washington, Wisconsin, and Wyoming.

In 9 States there are separate motor-vehicle departments, the administrative heads of which are known as commissioners of motor vehicles or registrars. The States of this group are California, Connecticut, Maryland, Massachusetts, Montana, New Hampshire, New Jersey, North Dakota, and Virginia.

There are 7 States in which the respective highway departments supervise motor-vehicle registrations and the collection of license fees in addition to their regular duties of constructing and maintaining the State highway systems. These are Arkansas, Oklahoma, Pennsylvania, Rhode Island, South Carolina, Texas, and West Virginia.

In the States of Alabama, Kentucky, New York, and Tennessee the State tax commissions have charge. In Florida and New Mexico the duty is performed by the

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[1] Report of the highway finance committee, Highway Research Board, National Research Council, presented at the annual meeting of the board at Washington, D. C., December 3, 1926.

State comptroller. In Idaho the bureau of law enforcement is responsible; in Mississisippi it is the auditor of public accounts; in Nebraska the department of public works; and in North Carolina the commissioner of revenue.

Although there is a State officer or State department responsible for the collection of the motor-vehicle license fees in each of the States, there are 16 States where the laws provide that certain county officers shall make the collections and distribute the registration plates. In 5 States—Iowa, Kansas, Nebraska, South Dakota, and Washington—the county treasurers make the direct collections. In Arizona, Idaho, and Nevada it is the county assessor who collects the license fees in the first place. The clerk of the county court is the collecting officer in Kentucky, South Carolina, and Tennessee. In Mississippi and Texas the county tax collector collects the motor license fees, together with the other local taxes. The probate judges are the local collecting officers in Alabama and the county sheriffs in Arkansas, and the State comptroller in Florida is authorized and directed by law to establish county agencies to collect the fees in each locality.

COLLECTION EXPENSES VARY WIDELY

Collection and administrative expenses.—The gross receipts from motor-vehicle registration fees, licenses, permits, fines, etc., and the amounts reported by the several States as spent for collection and administration are set forth in Table 2.

TABLE 2.—*Relation of collection and administrative expenses to gross receipts from license fees, etc.*

State	Gross receipts	Collection and administrative expenses	
		Amount	Per cent of gross receipts
Alabama	$2,511,129	-----	-----
Arizona	405,592	$18,000	4.5
Arkansas	3,150,000	12,000	.4
California	7,816,298	951,076	12.2
Colorado	1,430,299	71,515	5.0
Connecticut	5,644,247	-----	-----
Delaware	680,700	-----	-----
Florida	3,645,628	261,220	7.2
Georgia	3,010,415	98,297	3.3
Idaho	1,192,587	-----	-----
Illinois	12,969,754	-----	-----
Indiana	4,649,663	205,681	4.4
Iowa	9,741,103	713,036	7.3
Kansas	4,610,090	230,505	5.0
Kentucky	3,780,062	132,105	3.5
Louisiana	3,400,045	40,000	1.2
Maine	2,182,135	-----	-----
Maryland	2,576,301	250,000	9.7
Massachusetts	9,843,901	921,514	9.4
Michigan	14,526,002	300,000	2.1
Minnesota	9,744,834	-----	-----
Mississippi	1,530,000	45,900	3.0
Missouri	7,267,098	432,023	5.9
Montana	915,253	32,000	3.5
Nebraska	3,936,458	98,411	2.5
Nevada	209,197	10,584	5.1
New Hampshire	1,736,094	114,610	6.6
New Jersey	10,515,323	1,177,057	11.2
New Mexico	457,874	31,991	7.0
New York	25,506,245	-----	-----
North Carolina	8,359,844	149,761	1.8
North Dakota	1,083,573	150,000	13.8
Ohio	13,147,231	-----	-----
Oklahoma	4,576,572	-----	-----
Oregon	5,370,202	200,000	3.7
Pennsylvania	21,926,972	-----	-----
Rhode Island	1,863,955	306,492	16.5
South Carolina	2,366,076	187,729	7.9
South Dakota	2,445,112	21,511	.9
Tennessee	3,060,948	54,243	1.8
Texas	13,477,931	476,146	3.5
Utah	554,235	-----	-----
Vermont	1,497,146	82,037	5.5
Virginia	4,300,950	-----	-----
Washington	4,980,026	240,059	4.8
West Virginia	3,354,247	264,386	7.9
Wisconsin	7,896,210	380,000	4.8
Wyoming	482,857	-----	-----
District of Columbia	291,207	36,820	12.6
Total	260,619,621	8,696,709	-----

Thirty-four States and the District of Columbia reported that their collection and administrative expenses incurred on account of the registration and licensing laws amounted to $8,696,709 for that year. The total gross receipts for these same States which reported their collection and administrative expenses were $155,199,173. In these States, therefore, the collection and administrative expenses averaged 5.6 per cent of the total receipts. Among the individual States, however, there is a wide variation in this ratio, as shown in the table. In Arkansas the reported collection and administrative expenses were only $12,000, or 0.4 per cent, of the receipts; in Rhode Island they averaged 16.5 per cent. These are the two extremes.

In making such a comparison it must be borne in mind that certain items of expense are probably charged to this account in some States, whereas in other States similar expenditures are provided for by other funds or sources of revenue. In a number of States, as already pointed out, the actual collection of the fees and the registration of the motor vehicles is carried out by local county officers, such as sheriffs, county clerks, treasurers, or assessors. In such cases it is likely that a large part of the expenses connected with the administration of motor-vehicle licensing laws are borne by the local units of government, and that the reported expenses refer only to the administrative costs incurred by the State officials, who furnish the license plates and have but general supervision over the registration and licensing. In Tennessee, for instance, where the reported collection and administrative expenses amounted to only 1.8 per cent of the gross receipts, the county clerks collect the license fees from the car owners in their respective counties and charge each applicant in addition a fee of 50 cents, which the law authorizes as their compensation in full.[2]

In other States, where the ratio of collection and administrative expenses to gross receipts appear large and considerably above the average, services are rendered by State motor-vehicle departments and costs are incurred which are not found to the same extent in other States. In States like Maryland and Massachusetts, for example, the function of enforcing all provisions of the motor-vehicle laws relating to the use of the highways rests with the State motor-vehicle departments to a much greater degree than it does in many of the other States. Maryland's expense ratio is 9.7 per cent, and in Massachusetts it is 9.4 per cent. Somewhat similar conditions prevail in New Jersey, where the ratio is 11.2 per cent.

There are still other States in which the State departments are required to examine all applicants for licenses as to their ability and fitness to drive. Where such provisions are in effect and where the costs of this additional service are borne by the motor-vehicle departments and charged against the department's gross receipts, the ratio of expenses to receipts is naturally higher than in States where such services and costs are not involved.

A number of States, although permitting the deduction of collection and administrative expenses from the gross receipts, place limits upon expenses of this character. Arizona fixes the maximum at $25,000, North Dakota at $150,000. The California law provides that the motor-vehicle department's administrative expenses shall be kept within 20 per cent of the gross amount collected in license fees; Georgia limits it to 15 per cent; in New Mexico the limit is 5 per cent; and in North Carolina the administrative expenses may not exceed 10 per cent of the collections. The Rhode Island legislature has limited the State board of public roads which registers the motor vehicles and collects the license fees to an annual expenditure of $30,000 for plates, $89,000 for clerical salaries, and $30,000 for other expenses.

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[2] Motor Vehicle Laws of Tennessee, art. 5, sec. c., passed Apr. 16, 1919.

If the expense ratio which has been found to exist in these States, which have reported their respective collection and administrative expenses, is applied to the total gross license fee receipts of all of the States the resulting estimate of total collection and administrative expenses amounts to approximately $14,500,000. The remaining $246,100,000 is available for highway purposes and other uses. In most cases where no collection and administrative expenses are reported the legislatures make definite appropriations to meet such costs and all of the total receipts are turned over to the State treasurer's department, usually to the credit of certain highway funds.

In discussing these variations in administrative expense ratio it must be observed that one of the reasons for them is the variation in the fees charged. For example, no more labor or expense is necessary in registering cars where the license fees average $10 per car than where the average is but $5. The expense ratio will differ, however, in direct proportion to the amount of fees charged.

COLLECTION COSTS FROM 7 CENTS TO $3 PER CAR

The collection and administrative expenses of the States which reported this item amount to an average of 70.4 cents per motor vehicle registered. These States, in 1925, reported a total registration of 12,266,460 vehicles. When the collection costs are averaged over the number of cars registered in each of the reporting States wide variations are also noticed. In Arkansas these administrative expenses amounted to 7 cents per registered vehicle; that is the lowest. The highest average is found in Rhode Island, $3.02 per vehicle. It would appear that both of these averages are not only extreme but abnormal. The registration of automobiles in Rhode Island is in the hands of the State highway commission, and it is quite possible that the figures which were reported as the expense resulting from the administration of the motor-vehicle registration law may contain other items of expense incurred by the commission which pertain to other functions and duties of the commission.

TABLE 3.—*Relation of collection and administrative expenses to number of cars registered*

State	Number of cars registered	Collection and administrative expenses	
		Amount	Per car registered
Alabama	194,580		
Arizona	68,029	$18,000	$0.26
Arkansas	183,589	12,000	.07
California	1,440,541	951,076	.66
Colorado	240,097	71,515	.30
Connecticut	250,669		
Delaware	40,140		
Florida	286,388	261,220	.91
Georgia	248,093	98,297	.40
Idaho	81,506		
Illinois	1,263,177		
Indiana	725,410	205,681	.28
Iowa	659,202	713,036	1.08
Kansas	457,033	230,505	.50
Kentucky	261,647	132,105	.51
Louisiana	207,000	40,000	.19
Maine	140,499		
Maryland	234,247	250,000	1.07
Massachusetts	646,153	921,514	1.43

TABLE 3.—*Relation of collection and administrative expenses to number of cars registered*—Continued

State	Number of cars registered	Collection and administrative expenses	
		Amount	Per car registered
Michigan	989,010	$300,000	$0.30
Minnesota	569,694		
Mississippi	177,262	45,900	.26
Missouri	604,166	432,023	.72
Montana	94,656	32,000	.34
Nebraska	338,719	98,411	29
Nevada	21,169	10,584	.50
New Hampshire	81,498	114,610	1.40
New Jersey	580,554	1,177,057	2.00
New Mexico	49,111	31,991	.65
New York	1,625,583		
North Carolina	340,287	149,761	.44
North Dakota	144,972	150,000	1.04
Ohio	1,346,400		
Oklahoma	424,345		
Oregon	216,553	200,000	.93
Pennsylvania	1,330,433		
Rhode Island	101,756	306,492	3.02
South Carolina	168,496	187,729	1.11
South Dakota	168,028	21,511	.13
Tennessee	244,626	54,243	.22
Texas	975,083	476,146	.49
Utah	90,500		
Vermont	69,576	82,037	1.18
Virginia	282,650		
Washington	328,442	240,059	.73
West Virginia	217,589	264,386	1.22
Wisconsin	594,386	380,000	.64
Wyoming	47,711		
District of Columbia	103,092	36,820	.36
Total	19,954,347	8,696,709	

DISPOSITION OF LICENSE-FEE RECEIPTS

In the preceding tabulations a figure of $8,696,709 is used as representing the collection and administrative expenses incidental to the licensing of motor vehicles in 34 States and the District of Columbia. The reports, made by several other States indicated that certain expenditures were made for license-fee collection and other miscellaneous administrative purposes. As the two kinds of expense were not separated in the reports, these were not used in making the foregoing analysis. In determining what portion of the total gross receipts was used for highway and other related purposes in the following study all these reported expense items have been deducted, thus leaving balances available for other purposes. Excluding the District of Columbia it is found that the total gross receipts amounted to $260,328,414 and that the sum reported by the various States as the collection and administrative expense item was $11,955,927. Ten of the States did not report any administrative expense items for the reason that their motor-vehicle departments are maintained by definite appropriations from the general funds, thus making it unnecessary to make deductions from the license receipts in order to carry on the affairs of the departments.

After deducting the collection and administrative expenses as reported there is left a balance of $248,372,487. The disposition of this total amount is the subject of our further study. For the country as a whole it was as follows:

Purpose	Amount	Per cent of total
Construction and maintenance of State highway system	$177,452,200	71.5
Construction and maintenance of local roads	48,396,471	19.5
Principal and interest payments on State and local highway bonds	19,124,014	7.7
Transfers to general funds	478,183	.2
Expenses of highway departments	618,655	.2
Traffic control and patrol	770,391	.3
Other purposes	1,532,573	.6
Total	248,372,487	100.0

In studying this disposition of the motor-vehicle license receipts, it is significant to note that after a deduction of the reported administrative expenses 91 per cent of the balance is devoted directly to the construction and maintenance of highways; 71.5 per cent for State highway system, and 19.5 per cent for local roads. This 91 per cent of the available revenues for highway construction and maintenance amounts to $225,848,671, 79 per cent of which is spent on State highway systems and 21 per cent on local roads. Of these special revenues $19,124,014, or 7.7 per cent of the total amount available, was devoted to principal and interest payments on State and local highway bonds. The license fees devoted to other purposes constituted but 1.3 per cent of the total amount available.

HOW THE STATES DISTRIBUTE THE LICENSE RECEIPTS

The major legislative provisions relating to the distribution and disposition of the license fees derived from the registration and regulation of motor vehicles in the several States are as follows:

Alabama.—The expenses incident to the collection of the fees are first deducted. Of the balance, 80 per cent is credited to the State and 20 per cent is turned back to the incorporated city or town in which the owner of the registered vehicle resides. If the owner resides outside of an incorporated city or town the 20 per cent share is remitted to the county in which he resides. The statutes provide that the 80 per cent share credited to the State shall be used exclusively to create a sinking fund for the prompt payment of the principal and interest on the State road bonds outstanding, and for the construction, maintenance, and improvement of public highways, roads, and bridges. The statutes are silent as to the manner in which the incorporated municipalities and counties may use their shares of license fee revenues.

Arizona.—All the license fees are credited by the State treasurer to the State road-tax funds and out of this fund the legislature has appropriated a sum not to exceed $25,000 for the purpose of meeting the administration and collection expenses.

Arkansas.—A special fund is created in the State treasurer's office known as the State highway fund. All the money which the State receives from the collection of license fees and gasoline and oil taxes is credited to this fund. The State highway commission allots the sum of $3,000,000, or as much thereof as is available in the treasury by July 1 of each year, to the counties, apportioned on the basis of population. For the purpose of determining the use of these county-allotted funds the statutes classify the counties.

Class A includes 13 counties, named in the law, which are required to use the whole of the funds derived from the State in part payment of highway bonds and interest coupons. Class B includes only 3 counties which are required to devote 90 per cent of their State allotment to bond and interest payments and to place the remaining 10 per cent in the county highway improvement fund. Class C includes 11 counties which are required to devote 75 per cent of their State allotment to bond and interest payments and to credit the remaining 25 per cent to the county highway improvement fund. In class D there are 10 counties which are required to devote 50 per cent of their State allotment to bond and interest payments and the remaining 50 per cent to the county highway improvement fund. Class E includes 13 counties, the allotment being divided 25 per cent for bond and interest payments and 75 per cent credited to the county highway improvement fund. No counties are put in class F, of which the stated percentages are 10 per cent for bond and interest payment and 90 per cent for the county highway improvement fund. Class G contains 26 counties which are authorized to place all of their State allotments into their respective county highway improvement funds. The county courts are authorized to expend the funds put into the county highway improvement fund upon the public highways for construction and maintenance purposes.

California.—The motor-vehicle license fees are kept by the State treasurer in a motor-vehicle fund. From this fund are deducted the expenses of the motor-vehicle department, which may not exceed 20 per cent of the revenues. One-half of the balance in the motor-vehicle fund is paid over to the counties in proportion to the number of motor vehicles registered. The amount paid by the State for State inspectors and traffic officers is deducted from this county allotment. All amounts paid to the counties are credited to the road funds of the counties and are to be expended exclusively in the construction and maintenance of public roads, bridges, and culverts within the respective counties. The board of supervisors has the power to devote a portion of such allotted funds to the construction, maintenance, and repair of streets and bridges within incorporated cities. Annual reports have to be made by the county supervisors to the State department of public works showing the amount of money received and the disposition thereof. The balance in the State motor-vehicle fund is expended under the direction of the State department of public works for the maintenance, repair, widening, resurfacing, and reconstruction of State roads and highways. The funds allocated to the State department are not to be used for construction purposes though that use is permitted to the county highway officials.

Colorado.—The State treasurer deducts from the receipts derived from motor-vehicle license fees the necessary administrative expenses. Fifty per cent of the balance is expended by the State highway commission for the construction, maintenance, and improvement of the State highways and for no other purpose. The remaining 50 per cent is paid over to the counties, in proportion to the money collected, for the exclusive purpose of constructing, maintaining, and improving county roads and bridges.

Connecticut.—All license revenues are expended by the State highway commissioner for the construction, maintenance, or reconstruction of State highways. The law makes no mention of administrative expenses.

Delaware.—The State treasurer is directed to pay out of motor-vehicle revenues the interest on highway bonds and to put into a separate fund annually the necessary amount to meet the sinking-fund requirements. The balance is to be expended by the State highway department for the construction and maintenance of State highways.

Florida.—After deducting the collection and administrative expenses 5 per cent of the receipts from motor-vehicle license fees is set aside for the maintenance of the State road department; 25 per cent is allocated to the counties as a State-aid fund for the purpose of construction and maintenance of county roads. This amount is apportioned among the counties on the basis of the amount of fees collected. The balance, or 70 per cent, is placed in a fund to be used by the State road department for the construction and maintenance of State and Federal-aid roads.

Georgia.—The highway commission allocates the net amount of license fees collected to the counties according to post-road mileage. Such funds are expended by the highway commission in the building, repairing, and maintaining of public roads in each of the counties.

Idaho.—The county assessors who collect the license fees are directed to remit 10 per cent of their collections to the State treasurer who places these payments in the State highway fund. Ninety per cent of the fees collected accrue to the counties. The county commissioners of each county apportion to the interest and sinking fund of the county such amounts as are necessary for the current year. The balance of the fees is divided by the county commissioners among the regularly organized highway districts in proportion to the collections made from such districts. If any balances remain they are apportioned to the county road and bridge funds.

Illinois.—All license fees are deposited with the State treasurer and become a part of the State road fund. This fund is used for the purpose of meeting the principal and interest requirements on highway bonded indebtedness. If there is any balance left it is to be used for the improvement of the highways of the State or for the construction and maintenance of durable, hard-surfaced roads along designated routes.

Indiana.—After collection expenses are deducted all the receipts derived from motor-vehicle license fees are placed in a State highway fund which is available to the State highway commission for construction and maintenance purposes.

Iowa.—The law provides that 2½ per cent of the license fee collections shall be credited to a maintenance fund for the State highway commission and that 3½ per cent shall be credited to a maintenance fund for the motor-vehicle department. The balance, less 50 cents per license issued which is retained by the county treasurer to cover his registration expenses, 0.5 per cent of which is credited to a reimbursement fund, is turned into the primary road fund. This primary road fund is apportioned among the counties in the same ratio that the area of each county bears to the total area of the State. The law defines primary roads as those main marked roads which connect all the county seat towns with the main market centers. A small fund known as the Federal engineering fund is also created out of the license fees.

Kansas.—The county treasurers collect the license fees and remit to the Secretary of State 50 cents for each motor-vehicle license issued. Of the balance 25 per cent remains in the county where collected and is divided among the townships in proportion to the amount collected in each. The remaining 75 per cent of the fees is transmitted to the State treasurer and placed in the State highway fund, to which is also added the money derived from the gasoline tax. There is withdrawn annually from this State highway fund the sum of $300,000, which is placed in another fund known as the State-aid road fund and expended in the various counties upon the State highway system under the direction of the State highway commission acting in conjunction with the boards of county commissioners. Forty per cent of this sum is distributed equally among all of the counties of the State and 60 per cent is distributed in proportion to assessed valuation. The remainder of the State highway fund, after deducting the sum of $300,000 and a sum not to exceed $75,000 for the maintenance of the State highway commission, is distributed among the counties in the same proportion as noted above. The fund thus created in the various counties is known as the county and State road fund and is used for the construction, reconstruction, and maintenance of State roads in the counties. Not more than 20 per cent of this fund can be expended on county or township roads.

Kentucky.—All motor vehicle license fee receipts are deposited in the State road fund, out of which are paid the expenses of the State tax commission incident to the enforcement of the registration law. The balance of the fund is used only for the purpose of construction, reconstruction, maintenance, and repair of roads and highways.

Louisiana.—All license fees are deposited with the State treasurer and credited to the account of the general highway fund which is used by the State highway commission for the construction and maintenance of State highways.

Maine.—All the fees collected are turned over to the State treasurer and are used for the administration of the office and duties of the highway commission, including the expenses of administering the motor-vehicle department. The balance is used for the repair and maintenance of the State-aid highways under the direction of the commission and to meet the interest and principal payments of the bond issues for highway construction purposes.

Maryland.—The commissioner of motor vehicles is directed to remit to the State treasurer all license-fee revenues received by him to be placed in a special fund. Out of this fund the legislature's appropriations for the department's administrative expenses are met. The balance is distributed in quarterly installments 20 per cent to the city of Baltimore for use on its roads, 80 per cent to the State roads commission to be used for the oiling, maintenance, and repair of the State and State-aid roads.

Massachusetts.—All the motor-vehicle revenues received and collected by the registrar of motor vehicles are credited to a fund known as the highway fund Out of it the department's administrative expenses are paid. The balance is used under the direction of the highway division of the department of public works for maintaining, repairing, improving and constructing town and county roads; for maintaining, repairing and improving State highways and bridges; for construction of State highways partially financed through Federal-aid funds; for engineering services, purchase and maintenance of road-building machinery, snow removal, erection and maintenance of warning and direction signs, care of shrubs and trees on State highways; to meet the interest and sinking-fund payments on State highway bonds; to meet the State's share of the interest and sinking-fund payments on metropolitan park loans; and to meet the State's share of the cost of maintenance of boulevards in the metropolitan park district and the cost of construction of such boulevards. The distribution of funds for all of these purposes is left to the State commission except for those fixed or contractual payments which have to be met.

Michigan.—All license fees are deposited in the State treasury to the credit of the State highway fund. Six million dollars are returned to the counties in proportion to the amounts received as license fees from the owners of motor vehicles within the several counties. These funds are to be used for highway purposes under the jurisdiction of county authorities. Further appropriations from this State highway fund for 1925 were: (*a*) A sum not less than $1,200,000 to meet the interest and sinking-fund payments of State highway bonds; (*b*) $2,000,000 for the maintenance of State trunkline, Federal-aid and nontrunk-line highways; (c) $1,000,000 for building trunk-line bridges, including grade separations; (d) $500,000 for nontrunk-line highway maintenance and for nontrunk-line bridges; (e) the balance not otherwise appropriated for opening, widening, and improving State trunk-line and Federal-aid highways. There is also appropriated out of the State highway fund the sum of $300,000 to cover the operating expenses of the State highway department.

Minnesota.—The proceeds of the license fees and motor-vehicle taxes are deposited in the State treasury and constitute a fund known as the trunk-highway sinking fund. The law provides that out of this fund there shall be taken and deducted annually the amount required during the year as interest and principal payments on State highway bonds. After these payments are made or determined upon the balance is transferred from the trunk-highway sinking fund to the trunk-highway fund which is used by the State department of highways for administration, maintenance, and construction purposes. Up to the present time the State has not issued any highway bonds so it is not necessary to set aside any funds for interest and principal payments upon such bonds. The law also provides that the State shall reimburse the counties for certain work done by the counties on roads that are now a part of the State highway system. In fulfilling this requirement the State has made itself responsible for the interest and principal payments of such county bonds and there is set aside out of the trunk-highway sinking fund a sum amounting to $2,000,000 a year for these specific purposes.

Mississippi.—The county tax collector who collects the motor-vehicle license fees deducts 3 per cent as his commission, and turns the balance of his collections into the county road fund. These funds are used to maintain, repair, construct, or extend the highways in the county, preference being given to designated State highways.

Missouri.—All moneys arising from motor-vehicle license fees after deducting the administration and collection costs are deposited in the State treasury and credited to a State road fund. The revenues derived from the sale of option stamps are also turned into this fund. Out of this fund are paid: (a) The administrative cost and expenses of the State highway department; (b) $400,000 biennially for the purpose of providing the sum of not more than $25 per mile per annum for dragging and otherwise improving intercounty-seat highways; (c) the amount needed to match Federal-aid allotments; (d) the remainder to be used in the construction of State roads.

Montana.—After deducting from the receipts all administrative and collection expenses, 50 per cent of the balance is transmitted back to the counties from which the fees came and is deposited to the credit of the general road fund of the county; the remaining 50 per cent is deposited to the credit of the State highway fund, which is used for construction and maintenance purposes.

Nebraska.—The county treasurer of each county collects the motor-vehicle license fees. Thirty per cent of the fees are transmitted by the county treasurers to the State treasurer to be credited to the State highway fund and used for the maintenance of the State highway system. The remainder of the revenues is kept by the counties; 75 per cent to be used for the maintenance of that part of the State highway system within the respective counties and 25 per cent credited to the county's road dragging fund.

Nevada.—All motor-vehicle revenues are placed by the State treasurer in a motor-vehicle license fund. Disbursements from this fund are made as follows: (a) To meet the interest and principal payment requirements of the State highway bonds; (b) to meet the expenses of administering the registration law 50 cents per vehicle registered is placed in a motor-vehicle expense fund; and (c) after these requirements are met the State controller is directed to transfer the balance at the end of each three-month period to the State highway fund which is used by the State commission for the purpose of constructing, equipping and maintaining the highways designated as the State highway system.

New Hampshire.—After all expenses of the commissioner of motor-vehicles department have been met out of motor-vehicle revenues the balance is placed in a fund to be used by the State highway commissioner for the construction of State and State-aid roads and for their maintenance.

New Jersey.—The license-fee receipts, after making a deduction for the expenses of the motor-vehicle department, are deposited with the State treasurer to be used for construction and maintenance purposes by the State highway commission. The legislature at its last session appropriated $840,000 from the motor-vehicle funds and distributed this amount equally among the 21 counties. A further appropriation of $60,000 was made as aid in the maintenance of interstate bridges which had been freed by the Delaware River Free Bridge Commission.

New Mexico.—The law authorizes a collection fee of not more than 5 per cent, and the balance of the motor-vehicle revenues is deposited with the State treasurer. Two-thirds of this amount is credited to the State road fund to be used by the State highway commission for the maintenance of State highways in the several counties as near by as practicable in proportion to the fees paid by motor-vehicle owners residing in each county. The remaining third is transmitted to the county treasurers in proportion to the amount of fees paid and credited to the county road fund of each county for use in the maintenance of county roads.

New York.—Every month the collecting agency deposits 75 per cent of the funds on hand with the State treasurer and 25 per cent of the registration fees collected in each county with the treasurer of the county. The funds received by the State treasurer are used in the maintenance and repair of the improved roads of the State under the direction of the State commissioner of highways. All funds received by the county treasurers are used for the permanent construction and maintenance of town highways and county roads. The funds accruing to the city of New York are credited to the city's general fund.

North Carolina.—After the collection and registration expenses are deducted, which may not exceed 10 per cent of the total amount collected, the motor-vehicle license fees are credited to a fund for meeting interest, sinking fund and principal payment requirements of the bonds issued by the State for highway improvements. What balance is not needed for these specific purposes is used by the State highway commission for the maintenance of highways in the State system and for construction and reconstruction purposes. A provision of the law allows cities and towns to charge a license not to exceed $50 for any motor vehicle used in transporting persons or property for hire in lieu of all other charges, fees, and licenses.

North Dakota.—The moneys received into State treasury as motor-vehicle license fees are expended as follows: A sum not to exceed $150,000 per annum for expenses of the State highway commission for carrying out provisions, of the motor-vehicle registration act and the State highway commission act; a further sum of $130,000 annually to meet State obligations for bridge loans; 50 per cent of balance paid to county road funds for the construction and maintenance of county roads; the remaining 50 per cent shall remain in State highway fund for construction, reconstruction, and maintenance of Federal-aid and State-aid roads.

Ohio.—Fifty per cent of the license fees collected is turned back for the use of the municipal corporation or county in which the fees are originally collected. Such moneys constitute a fund which is used for the maintenance and repair of public roads, highways and streets and for no other purpose. The remaining 50 per cent of the receipts is deposited in the State treasury and credited to a State maintenance and repair fund. The expenses incident to the administration of the registration law are paid out of this fund and the remainder is for the use of the director of highways and public works.

Oklahoma.—An amount equal to 40 per cent of the revenues derived from license fees is credited to the State highway construction and maintenance fund; 60 per cent is transmitted to the counties in proportion to the amount collected in each respectively. This fund is used for construction and maintenance. The county treasurers are directed to turn over to the treasurers of incorporated cities and towns 15 per cent of the fees collected from motor vehicle owners residing within such incorporated places which amount is credited to the local street and alley fund. The balance of the county's share is credited to the county highway construction and maintenance fund.

Oregon.—After deducting the collection and administrative expenses the revenues derived from registration fees are divided between the State and the counties, 75 per cent of this balance is transferred to the State highway fund, 25 per cent is transmitted to the county treasurers of the various counties in proportion to the receipts from each county. In each county there is created a motor-license fund arising from the money received from the State. This county fund is used and disbursed in the following order: (*a*) Payment of interest and principal of county highway bonds; (*b*) where no bonds have been issued or where other provision has been made for the interest payment and retirement of such bonds, the funds are used to prepare the roadbeds and bridges for the the construction of State highways; (*c*) if provision has been made for meeting bond requirements and for the payment of expenses of preparing the roadbed then the money may be used in the discretion of the county court for general road improvements.

Pennsylvania.—The receipts derived from the licensing and registering of motor vehicles are placed in a fund known as the motor-license fund. Out of this fund are paid all the administrative expenses of the State highway department which is responsible, along with its other duties, for the collection of the license fees. A prior lien on this fund is the amount of money necessary for the payment of interest and sinking-fund charges on highway bonds issued by the State. The rest is at the disposal of the highway department for the purpose of assisting in the maintenance, construction, reconstruction, and improvement of State and State-aid highways, and also for the maintenance of interstate bridges.

Rhode Island.—All the motor-vehicle revenues are deposited in the State treasury and out of them are paid the costs of license plates not to exceed $30,000, the annual salaries of clerks employed by the State board of public roads not to exceed $89,000, and other expenses not to exceed $30,000. The balance is used for the construction, repair, and maintenance of, and removal of snow from State roads and highways under the direction of the State authorities.

South Carolina.—The receipts from motor-vehicle license fees are expended by and under the direction of the State highway department for the construction and maintenance of the State highway system.

South Dakota.—The collections are made by the county treasurers who forward one-half of the receipts to the State treasurer to be placed in the State highway fund for construction and maintenance purposes; 2 per cent is transmitted to the secreatry of state to be placed in the State motor-vehicle fund for the purpose of meeting his expenses incurred in connection with the registration law; and 48 per cent is credited to the motor-vehicle fund of the county in which the money is collected. The law further provides that 75 per cent of the funds in the State highway fund shall be apportioned for expenditure among the counties on the basis of assessed valuation; the remaining 25 per cent, after the commission's expenses are deducted, is used by the State highway commission for highway purposes.

Tennessee.—Fifty per cent of the net receipts is distributed equally among the counties of the State and the remaining 50 per cent is expended upon such State highways for construction and maintenance as may be designated by the highway commission. The State highway commission also has the right to designate the roads in the several counties upon which the counties are to spend their respective shares of the funds and the character of construction and maintenance.

Texas.—The motor vehicle license fees are based upon two separate computations; 17½ cents per horsepower is one part of the license fee, the other is the result of applying a certain rate to the weight of the vehicle. The county collector of taxes who collects the license fees is directed to deposit to the credit of the county road and bridge fund an amount equal to 17½ cents per horsepower for every vehicle registered. The balance of the fees derived from the weight factor is transmitted and credited to the State highway department. Approximately three-fourths of the total collections is turned over to the State department and one-fourth is retained by the counties.

Utah.—All the fees are deposited with the State treasurer and credited to a fund known as the motor-vehicle registration fund. Out of this fund are paid the registration expenses. The balance is applied to interest and principal payments on State road bonds.

Vermont.—The net fees are turned over to the State treasurer and are made available for road maintenance by the State highway department.

Virginia.—After deduction of registration and collection expenses the motor-vehicle revenues are deposited in the State treasury and are expended by the highway commission for the construction and maintenance of State highways and bridges.

Washington.—The State treasurer places all fees to the credit of the motor-vehicle fund in the State treasury from which are paid annually: Expenses of the motor-vehicle department for issuing licenses; repayments to counties entirely surrounded by water; the sum of $1,400,000 to the credit of a State-treasury fund called "primary highway maintenance fund"; the balance to paving and general road construction as provided by appropriation.

The primary highway maintenance fund is distributed as follows:

(1) To each first and second class city in which are streets forming a part of the primary State highway, a sum equal to $500 per mile for each primary highway in such city for maintenance and improvement of streets.

(2) To each third and fourth class city, similarly to above, $300 per mile.

(3) To each county in which are located primary highways, similarly, $300 per mile.

(4) The balance to the counties in proportion to money paid into the highway fund for maintenance of primary highways.

West Virginia.—The collection of license fees and the registration of motor vehicles are under the jurisdiction of the State road commission. The receipts are placed in the State road fund. Out of it are paid the commission's expenses and the balance is devoted to: (*a*) Principal and interest payments on State highway bonds; (*b*) the maintenance and construction of State roads.

Wisconsin.—After deducting collection and administrative expenses the net motor-vehicle license fees and the net gasoline-tax receipts and Federal-aid allotments are appropriated for the use of the State highway commission under the following restrictions and directions of the legislature:

(1) One hundred thousand dollars allocated annually to meet the expenses of the highway commission; (2) after the commission's expenses are set aside, 10 per cent of the remainder of the appropriation is devoted to the improvement of the State trunk highway system to be expended on such projects as the commission shall from time to time determine; (3) $100,000 is set aside annually for the marking and signing of the State trunk highway system; (4) the counties receive from $300 to $500 per mile for the maintenance of the State trunk highway system, the cities receive similar payments per mile of streets forming State trunk highway connections, and special sums are provided for, to be paid counties, cities, and villages which provide and maintain free swing or lift bridges on the State trunk-highway system; (5) $250,000 is appropriated annually to pay the State's portion of the cost of bridges; (6) $25,000 for improving highways forming connections between State charitable institutions and the State trunk-highway system; (7) $50,000 for the construction and maintenance of roads in State parks; (8) $15,000 for the purpose of making surveys of local road materials; (9) for the improvement of roads and streets not part of the State trunk highway system in towns, villages, and cities (each town and village receives $25 per year for each mile of such streets, cities with a population of not more than 10,000 receive $50 per mile, cities with a population from 10,000 to 39,000 receive $100 per mile, cities with a population from 40,000 to 150,000 receive $150 per mile, and cities with a population in excess of 150,000 receive $200 per mile); (9) the balance remaining after the amounts for the above-named pruposes are appropriated is used for the improvement of the State trunk highway system and the county trunk highway systems.

Wyoming.—The revenues derived from motor-vehicle license fees are devoted to the interest and sinking-fund payments on State highway bonds.

PART II. RECEIPTS FROM GASOLINE TAXES

As has already been indicated the gasoline tax continues to furnish a more and more important source of funds derived from motor-vehicle owners and operators used for highway purposes. In 1925 there were only four States—Illinois, Massachusetts, New Jersey, and New York—which did not resort to the gasoline tax as a revenue-producing measure for the construction and maintenance of highways. The tax rate ranges from 1 to 5 cents per gallon. In 1925 a total tax of $146,028,940 was collected from the sale of 6,457,783,284 gallons of gasoline used in motor vehicles, from which it appears that the average tax rate is 2.2 cents per gallon. In the four States mentioned above which did not collect this type of tax, and in other States in which a gasoline tax was not in effect during the whole 12-month period, there was consumed by motor vehicles, according to the best estimates that can be made, 2,131,056,365 gallons of gasoline which was not taxed.

The gasoline tax is a comparatively new method of raising funds for highways. Although a form of sales tax it is generally conceded that the gasoline tax is in essence a type of road toll and that its yield and earnings reflect more or less directly the use made of the highways by motor vehicles and measure to a certain degree the benefits resulting therefrom.

Point of collection.—There are only three States—Colorado, North Carolina, and Pennsylvania—which do not collect the tax at the source. These States gather the tax from the dealer who sells to the consumer. In all other States the tax is collected at the source. This usually means the collection from the person or corporation which makes the original sales in the States. In most of the laws they are termed manufacturers, importers, distributers, or wholesalers.

Collection period.—In all except four of the States which have in effect a gasoline tax the collections are made monthly. In California, Georgia, Pennsylvania, and Tennessee the gasoline taxes are paid to the respective States quarterly. In most cases where the tax is collected at the source, the tax is remitted by the wholesale distributers accompanied by sworn statements showing in detail the receipts and sales.

Refunds.—The laws of 28 States provide for a refunding of the tax paid on gasoline sold for other purposes than for automobile use. The Indiana law, for example, has this provision: "* * * the tax does not apply to gasoline purchased for use in operating stationary gas engines, tractors used for agricultural purposes, motor boats, aeroplanes or aircraft, or for use in cleaning or dyeing, or for any other commercial use." Whether the tax is paid by the wholesaler or by the dealer, gasoline used for these purposes has, in most cases, been taxed when sold to the consumer and the consumer in the first instance pays the tax which is included in the price he pays for the gasoline. Where gasoline is sold under such circumstances, there is usually a provision made in the law whereby the consumer can secure a refund of the amount of taxes paid upon gasoline not used in motor vehicles by presenting a sworn statement of his purchase accompanied by invoices or purchase memoranda. There are still 16 out of the 44 States which collect the tax which have not made any provision for refunds and which tax all the gasoline sold irrespective of its use. The 28 States which grant refunds collected, during 1925, $100,604,852 and refunded $4,109,513, or 4.1 per cent. It is no doubt true that a great many persons buying and using but small quantities of gasoline for purposes other than highway use never take the trouble to secure any refunds. How great that number is can not be determined definitely. There may also be a few who through false statements or otherwise secure for themselves refunds to which they are not entitled. The gross amounts collected and refunded in the 28 States which provide for refunds are set forth in Table 4.

TABLE 4.—*Gasoline tax receipts and refunds in 1925 by the 28 States providing for refunds*

State	Total gasoline tax receipts	Amount of refunds	Ratio of refunds to total receipts	Net gasoline tax receipts
			Per cent	
Arizona	$1,035,551	$179,600	17.3	$855,951
Arkansas	3,230,559	280,199	8.7	2,950,360
California	16,150,387	1,193,598	7.4	14,956,789
Colorado	1,991,531	30,585	1.5	1,960,946
Delaware	350,580	8,499	2.4	342,081
Idaho	932,064	36,621	3.9	895,443
Indiana	7,832,462	179,413	2.3	7,653,049
Iowa	3,568,184	63,069	1.8	3,505,115
Kansas	3,000,253	95,059	3.2	2,905,194
Maine	1,283,874	15,526	1.2	1,268,348
Maryland	2,022,986	45,950	2.3	1,977,036
Michigan	8,742,392	506,314	5.8	8,236,078
Minnesota	3,989,282	125,342	3.1	3,863,940
Missouri	4,234,070	74,955	1.8	4,159,115
Nebraska	2,202,236	8,434	0.4	2,193,802
Nevada	335,446	16,741	5.0	318,705
New Hampshire	716,140	9,068	1.3	707,072
North Carolina	6,238,508	156,130	2.5	6,082,378
North Dakota	649,416	15,000	2.3	634,416
Ohio	9,133,785	123,835	1.3	9,009,950
Oregon	3,065,151	156,056	5.1	2,909,095
South Carolina	3,870,588	5,185	0.1	3,865,403
South Dakota	2,122,406	274,808	12.9	1,847,598
Virginia	3,863,117	161,166	4.2	3,701,951
Washington	3,205,114	184,302	5.8	3,020,812
West Virginia	2,222,329	35,590	1.6	2,186,739
Wisconsin	4,155,469	123,793	3.0	4,031,676
Wyoming	460,972	4,675	1.0	456,297
Total	100,604,852	4,109,513	4.1	96,495,339

GASOLINE TAX REFUNDS VARIABLE

The ratio of the refunds to the total collections varied a good deal in the several States. The lowest amount refunded relative to the total gasoline-tax receipts is found in South Carolina, where the ratio is 0.1 per cent; the highest in Arizona, where it is 17.3 per cent. In South Dakota the refunds were 12.9 per cent of the collections. In both of the latter States unusual conditions are found with respect to these refunds. It has been intimated by some one that the refunds in Arizona were relatively high because taxes were paid on certain amounts of gasoline which were afterwards shipped outside of the State, which resulted in claims having been put in for refunds. The reasonableness and justice of exempting from taxation gasoline not used by motor vehicles can be readily admitted, in as much as the prime purpose of the tax is to secure funds from users of the highways in proportion to the use.

Collection costs.—Only 21 States reported separately the cost of collecting gasoline taxes. In these States the gross amount collected in 1925 was $82,456,346, and collection costs as reported were $217,393. The collection costs, therefore, amounted to an average cost for these States of $2.64 per thousand dollars, or 0.264 per cent. The details are set forth in Table 5.

TABLE 5.—*Costs of gasoline tax collection*

State	Gross amount collected	Total number of gallons taxed	Reported collection costs	Collection costs per $1,000 collected	Collection cost per million gallons taxed
Alabama	$2,140,802	107,040,092	$9,461	$4.42	$88.39
California	16,150,387	747,839,462	7,393	.46	9.89
Florida	7,657,507	210,323,517	6,000	.78	28.53
Georgia	4,418,824	138,802,152	4,200	.95	30.26
Idaho	932,064	30,809,320	9,466	10.16	307.24
Indiana	7,832,462	272,980,870	12,436	1.58	45.56
Iowa	3,568,184	175,255,740	5,520	1.55	31.50
Maine	1,283,874	56,513,741	5,596	4.36	99.02
Maryland	2,022,986	98,851,813	2,500	1.24	25.29
Michigan	8,742,392	411,803,894	41,358	4.73	100.43
Mississippi	2,494,274	83,142,469	1,800	.72	21.66
Missouri	4,234,070	207,955,474	23,429	5.53	112.66
Nebraska	2,202,236	109,690,122	4,963	2.25	45.25
New Mexico	537,356	20,490,892	26,868	50.00	1,311.22
Oregon	3,065,151	96,969,835	6,553	2.14	67.58
Tennessee	3,407,886	122,000,680	22,768	6.68	186.62
Utah	1,064,004	32,217,216	3,750	3.52	116.40
Virginia	3,863,117	123,398,365	5,604	1.45	45.41
West Virginia	2,222,329	76,331,660	7,500	3.37	98.26
Wisconsin	4,155,469	201,583,789	10,000	2.41	49.61
Wyoming	460,972	20,746,056	228	.49	10.99
Total	82,456,346	3,344,747,159	217,393	[1] 2.64	[1] 65.00

[1] Average.

It is noted that the collection costs vary materially in this group of States which represent approximately half of the number of States levying a gasoline tax. In California the reports show that it costs the State only 46 cents to collect $1,000 of gasoline taxes; in New Mexico, which is at the other extreme, the collection costs amount to $50 per $1,000 collected. These differences in collection costs can be explained in several ways. It is quite probable that some of the States in arriving at these cost figures include only such extra expenditures incurred by those State officers whose duty it is to collect the gasoline tax which are over and above the office expenses prior to the assumption of the new duty. In New Mexico, on the other hand, the law provides that the collection expenses shall not exceed 5 per cent of the collections and, according to the reports, the collection costs come right up to this maximum limit fixed by statute. There is another consideration which should have an effect on this ratio: It should not involve any greater costs to collect a tax of 2 or 3 cents per gallon than a tax of 1 cent per gallon. It would, therefore, appear that if the States followed a uniform system of cost accounting, the collection costs per $1,000 would be correspondingly lower in the States charging the higher gasoline tax rates.

In Table 5 the collection costs are also computed on a per-million-gallon basis. In making that comparison of costs the element of the tax rate is completely eliminated. The costs range from $9.86 per million gallons in California to $1,310 in New Mexico, the average for this group of States being $65 per million gallons.

Aside from New Mexico the costs of collecting this amount of revenue are low when compared with the collection costs of other forms of revenue. In 1924 the collection of the custom revenues cost the Federal Government $22.50 per $1,000; the internal-revenue collection costs were $12.10 per $1,000. These figures are compared with the collection cost of $2.64 per $1,000 of gasoline taxes received. The collection cost of these Federal taxes per $1,000 amounts to over five times the collection cost of the gasoline tax.

DISPOSITION OF THE GASOLINE-TAX REVENUES

The gasoline-tax revenues in 1925 amounted to $144,921,949 after deducting $217,393 which 21 States reported as their total collection costs. In addition to this amount $889,598 was collected in the District of Columbia. In the analysis which follows these District of Columbia revenues are not included. The disposition of the gasoline-tax revenues was made for the following purposes in the amounts indicated.

Purpose	Amount	Per cent
Construction and maintenance of State highway system	$97,084,753	67.0
Construction and maintenance of local roads	31,848,464	22.0
Principal and interest payments on State and local bonds	5,853,703	4.0
Transfers to general fund	3,722,796	2.6
Use on city streets	3,097,892	2.1
Other purposes	3,314,341	2.3
Total	144,921,949	100.0

According to this classification 89 per cent of the net gasoline-tax receipts is used directly for the construction and maintenance of rural highways, 67 per cent being devoted to State highway systems and 22 per cent to the construction and maintenance of local roads. The remaining 11 per cent is also used for the most part for road and street purposes.

Disposition of gasoline-tax revenues by States.—The statutes of the several States make definite provision for the disposition of the gasoline-tax revenues just as in the case of the receipts from license and registration fees analyzed in Part I. The laws of some of the States provide that the money derived from the gasoline tax shall be distributed and applied in the same manner as the license fees; in other States a totally different policy is followed. In the analysis of the statutory provisions made by the different States with respect to the disposition of gasoline-tax revenues which follows, these similarities and dissimilarities of policy are set forth.

Alabama.—The tax is collected by the State tax commission, placed in the State treasury, and all receipts are distributed equally every month among the 67 counties, to be used only for maintenance and repair of roads and bridges.

Arizona.—The Secretary of state collects the tax and after deducting collection expenses deposits the money with the State treasurer to the credit of the gasoline-tax fund. One-half of this fund is turned over to the State road-tax fund and the other half is turned back to the counties in proportion to the amount collected in each to be used for the maintenance of county roads.

Arkansas.—All gasoline and oil tax revenues are deposited in the State highway fund. The distribution is set forth in detail in the section of this article devoted to the disposition of motor-vehicle revenues.

California.—The gasoline-tax revenues are deposited in the motor-vehicle fund, together with the license fees. The disposition of this fund is set forth in the section of this article devoted to the disposition of motor-vehicle revenues.

Colorado.—Fifty per cent of the receipts from the gasoline tax is deposited to the credit of the State highway fund to be used for the construction, maintenance, and improvement of the State highways. The remaining 50 per cent is apportioned among the several counties according to the mileage of State routes and State highways.

Connecticut.—The gasoline-tax revenues are expended in the same manner as the license fees by the State highway commissioner for the construction, maintenance, or reconstruction of State highways.

Delaware.—The State uses the gasoline-tax revenues in the same manner as the motor-vehicle license fees.

Florida.—Three-fourths of the gasoline-tax receipts are placed in the State road fund for the construction and maintenance of State and Federal-aid roads; the remaining fourth is divided equally among the counties to be used for construction and maintenance purposes.

Georgia.—Of the 3½-cent tax collected for each gallon of gasoline sold, the law provides that 1½ cents shall be allocated to the State highway department to be used for matching Federal aid; 1 cent is turned back to the county treasurers on the basis of the county mileage of the State-aid road system to be used for construction and maintenance; and 1 cent is set aside to meet the deficiency in the State treasury arising out of the deficiency of the rental of the Western Atlantic Railroad.

Idaho.—The balance after administrative expenses have been paid is placed in the State highway fund. The distribution of this fund is set forth in the section of this article devoted to the distribution and division of the motor-vehicle license fees.

Illinois.—No gasoline tax is in effect.

Indiana.—The law provides that the gasoline-tax revenues after the collection expenses have been deducted shall be deposited in the State highway fund; two-thirds of the net receipts are used by the State highway commission for construction and maintenance, and one-third is transferred to the general fund and credited to a special fund known as county, cities, and town gasoline fund. Three-fourths of this special fund are distributed as follows: 50 per cent is divided equally among the counties and 50 per cent is divided among the counties in proportion to the mileage of free gravel, macadam, and county-unit roads. The counties are required to use the funds they receive for the construction and maintenance of roads. The remaining fourth of this special fund is distributed among the cities and incorporated towns of the State in proportion to population and is to be used for the maintenance and repair of streets, preference being given to those streets connecting State highways.

Iowa.—The collection expenses are paid out of the gross receipts. The balance is divided as follows: One-third is credited to the primary road fund, the distribution of which is described in the license fee section; two-thirds is apportioned among the counties on the basis of area. One-half of this fund apportioned among the counties is placed in a county road fund and one-half is placed by each county in a township road fund on the basis of township road mileage.

Kansas.—The gasoline-tax receipts are placed in the State highway fund. The manner in which this fund is apportioned and used is set forth in the motor-vehicle license fee section of this article.

Kentucky.—Gasoline-tax revenues are used only for the purpose of construction, reconstruction, maintenance, and repair of roads and highways and for interest and sinking-fund payments of State bonds issued for highway purposes.

Louisiana.—The gasoline-tax revenues are credited to the general highway fund and are allotted for the exclusive purpose of the upkeep, repair, and maintenance of the State highways and bridges.

Maine.—The gasoline-tax receipts are placed in the State highway fund and are used in the following manner: 16⅔ per cent for the maintenance of State highways and bridges and State-aid highways; 16⅔ per cent for the construction of third-class highways; 33⅓ per cent for the construction of State highways; and 33⅓ per cent for the construction of State-aid highways.

Maryland.—The gasoline-tax receipts are used for the maintenance and reconstruction of the roads of the State highway system.

Massachusetts.—No gasoline tax is in effect.

Michigan.—The revenues derived from the gasoline tax are used for the following purposes: (*a*) To meet deficiencies in appropriations owed to several counties on State-award highways; (*b*) for the payment of interest on State highway bonds; (*c*) the balance is used for the general construction, improvement, and betterment of the public highways within the State.

Minnesota.—The gasoline-tax revenues are credited to the trunk-highway fund. The manner in which this fund is expended is described in the preceding section relating to the disposition of motor-vehicle license fees.

Mississippi.—The gasoline-tax revenues are placed in the State highway fund. Fifty per cent of the revenues is used for the maintenance of the highway department and for the construction and maintenance of State highways and bridges. The remaining 50 per cent is returned to the counties in proportion to the number of motor vehicles registered and is used for the construction and maintenance of county roads, bridges, and culverts.

Missouri.—All the money derived from the gasoline tax is expended by the State highway commission for the construction and maintenance of State highways.

Montana.—The gasoline-tax receipts are apportioned in the following manner: (*a*) 15 per cent is credited to the State highway fund; (*b*) 55 per cent is divided equally among the counties; and (*c*) 30 per cent is turned into the general fund of the State.

Nebraska.—The revenues derived from the gasoline tax are placed in the gasoline highway fund, which is used for the construction and maintenance of State and Federal aid highways.

Nevada.—All the gasoline-tax receipts are deposited in a gasoline-license tax fund. Fifty per cent of this fund is transferred to the State highway fund to be used for construction and maintenance purposes; 50 per cent is transferred to the State highway fund of the various counties in proportion to the number of registered motor vehicles.

New Hampshire.—The gasoline-tax revenues are used in the same manner and for the same purposes as the motor-vehicle license fees.

New Jersey.—No gasoline tax is in effect.

New Mexico.—The gasoline-tax revenues are placed in the State road fund and are used for the maintenance of State highways.

New York.—No gasoline tax is in effect.

North Carolina.—The gasoline-tax revenues are credited to the State highway fund and are disposed of by the State highway commission in the following manner: (*a*) $250,000 is set aside to meet the expenses of the State highway commission; (*b*) a sum is appropriated sufficient to meet the annual interest charges on the $65,000,000 bond issue for highway purposes; (*c*) a sum is appropriated sufficient to create $500,000 sinking fund annual payments upon the $65,000,000 bond issues; and (*d*) the balance is used by the State highway commission for the construction and maintenance of roads.

North Dakota.—All the revenues derived from the gasoline tax with the exception of $200,000 are deposited in the State highway fund for the construction, reconstruction, and maintenance of the Federal-aid and State-aid highways.

Ohio.—The law provides that the gasoline-tax receipts shall be distributed and disposed of by the State treasurer in the following manner: (*a*) The first $50,000 collected is placed by the State treasurer in a gasoline rotary tax fund to which sums are regularly added to keep this fund floating at $50,000; (*b*) 30 per cent of the balance is paid out to State municipal corporations, in proportion to the number of motor vehicles within such municipalities during preceding calendar year, for street maintenance purposes; (*c*) 25 per cent is divided equally among the counties for maintaining and repairing county road systems; and (*d*) 45 per cent is apportioned to and expended by the State highway department for the purpose of maintaining, repairing, and keeping in passable condition the roads and highways of the State.

Oklahoma.—One-third of the gasoline-tax revenues is credited to the highway fund of each county and is apportioned to each county in that percentage which the population and area of the county bear to the population and area of the State, to be used for the construction and maintenance of county or township roads; two-thirds of the receipts are deposited to the credit of the State highway construction and maintenance fund.

Oregon.—The gasoline-tax revenues are all placed in the State highway fund to be used for construction and maintenance purposes on the State highway system.

Pennsylvania.—The money derived from the gasoline tax is distributed in the following manner: One-fourth is credited to the counties in proportion to the amount of taxes collected and is used for the construction and maintenance of highways and for the payment of interest on county road bonds; three-fourths are deposited in the State motor-license fund and used to meet interest and sinking-fund payments on State highway bonds; and the balance is used for construction and maintenance purposes.

Rhode Island.—The gasoline tax revenues are placed in the State highway fund and are used for the construction and maintenance of secondary roads.

South Carolina.—Three-fifths of the gasoline-tax revenues are credited to the State highway department, and two-fifths are distributed among the counties on the basis of the amount of license fees collected, for the construction and maintenance of roads not in the State highway system.

South Dakota.—The receipts derived from the gasoline tax are placed in the State highway fund and are expended by the State highway commission for the construction, reconstruction, and maintenance of highways and roads under the commission's jurisdiction.

Tennessee.—The gasoline-tax revenues are all used for the construction and maintenance of highways of the State.

Texas.—The receipts derived from the gasoline tax are apportioned as follows: (*a*) One-fourth to the free school fund; (*b*) three-fourths to the State highway fund for construction and maintenance purposes.

Utah.—The gasoline-tax receipts are placed to the credit of a fund for the payment of interest and sinking-fund charges on State road bonds.

Vermont.—The gasoline-tax revenues are available the same as the motor-vehicle license fees for road maintenance by the State highway department.

Virginia.—The receipts derived from the gasoline tax are disposed of in the following manner: (*a*) 2 cents of the rate is appropriated for the construction of roads and projects on the State highway system; (*b*) 1 cent is devoted to the construction of roads and bridges on county highway systems distributed among the counties on the basis of the total State taxes paid by counties into the State treasury; (*c*) the extra 1½ cents, added to the former rate in 1926, is turned over to the counties if they match the amount by an appropriation of their own.

Washington.—All the gasoline-tax receipts are placed in the motor-vehicle fund, the disposal of which is set forth in the preceding section devoted to the disposition of motor-vehicle license fees.

West Virginia.—The revenues derived from the gasoline tax are disposed of in the same manner as the revenues derived from motor-vehicle license fees.

Wisconsin.—The gasoline-tax revenues, together with the motor-vehicle license fees, are all placed in a fund from which the annual appropriations for State highway purposes are made. These have been set forth in detail in the preceding section.

Wyoming.—The receipts derived from the gasoline tax are apportioned among the counties in proportion to the number of miles of State highway in each and used for maintenance purposes.

GAS TAX DISTRIBUTION LIKE LICENSE FEE USE IN 14 STATES

In 14 of the 44 States which have in effect a gasoline tax there is no distinction made between the revenues derived from this kind of tax and the revenues produced by the license fees, so far as the distribution and use of these funds are concerned. They are: Arkansas, California, Connecticut, Delaware, Idaho, Kansas, Kentucky, Louisiana, Minnesota, Rhode Island, Vermont, Washington, West Virginia, and Wisconsin. The remaining 30 States provide for a disposition of the gasoline tax receipts different from that of the motor-vehicle license fees.

According to the reported figures a somewhat larger proportion of the gasoline-tax revenues is used for the construction and maintenance of local roads than is found in the case of motor-vehicle license fee receipts. In 1925, 22 per cent of net gasoline-tax revenues was devoted to local highway purposes; the proportion of the motor-vehicle license fees for that purpose amounted to only 19.5 per cent. The sum of money taken out of license fees for local highway purposes, however, was greater; $48,396,471 derived from motor vehicle license fees was used for local highway purposes and $31,848,464 derived from gasoline taxes.

In only eight States is there any direct provision in the law which calls for an apportionment of any of these motor-vehicle revenues or gasoline taxes among cities or municipalities. These States are Alabama, Indiana, Maryland, Ohio, North Carolina, Oklahoma, Washington, and Wisconsin. The laws in these States provide that certain portions of the funds which the State receives shall be allocated to municipalities and shall be used by them for street improvements. These provisions have already been covered in the digest of the laws of the various States pertaining to the distribution of the license fees and gasoline taxes. It is believed, however, that special attention should be called to this particular phase of the disposition of motor-vehicle revenues at this point because of the rather wide interest given to this phase of the subject at this time.

In Alabama 20 per cent of the balance from motor-vehicle license fees (after deducting collection expenses) is turned back to the incorporated city or town in which the owner of the registered vehicle resides.

The gasoline-tax law of Indiana provides that a certain portion of the net revenues derived from the gasoline tax shall be distributed among the cities and incorporated towns of the State. The details relating to the apportionment of the gasoline-tax receipts are set forth in that part of the gasoline-tax section devoted to Indiana. The net result is that 8⅓ per cent of the net gasoline-tax collection is distributed among the cities and incorporated towns of the State for the maintenance and repair of their streets. In using this money preference is to be given to those streets connecting State highways. In 1925 the cities and incorporated towns of the State were entitled to receive according to these provisions $636,684.

The Maryland law makes direct provision that 20 per cent of all the license fees collected in the State shall be paid over to the city of Baltimore for use on its streets. The number of motor vehicles registered in the city of Baltimore in 1925 was 44 per cent of the total number of vehicles registered in the whole State of Maryland. It would appear therefore that the city of Baltimore had returned to it approximately half of the registration fees collected within its limits. This conclusion is obviously based on the assumption that the average license fee per vehicle collected in Baltimore does not differ materially from the average license fee collected throughout the State. No part of the gasoline-tax revenues is apportioned to Baltimore nor do any of the other cities and municipalities in the State share in either the license-fee or gasoline-tax receipts.

The Wisconsin law provides that a portion of both the license-fee receipts and the gasoline-tax revenues shall be paid over to towns, villages, and cities for the improvement of roads and streets not part of the State and county trunk highway systems. The payments are based upon the amount of mileage in the municipalities and the amount paid per mile varies according to the size and population of the municipality. The details are fully set forth in the motor-vehicle license fee section. For the fiscal year ended June 30, 1926, according to the report of the Wisconsin Highway Commission, the net amount of license fees and gasoline taxes collected by the State was $13,279,529. Of this amount $1,865,218, or 14 per cent, was turned over to towns, villages, and cities. The mileage upon which these payments were based amounted to 62,743 miles. The towns received $1,422,854, or 76 per cent, of the total amount of money distributed in this manner, and they contained 90 per cent of the total mileage. Villages received $37,540, or 2 per cent, of the amount distributed. There were 161 cities in the State, of various sizes, which received $404,824, or 21.6 per cent, of this amount, based upon 4,330 miles of streets, which was but 6.9 per cent of the total mileage upon which these payments to localities were based. For the 161 cities which participated in these revenues, the average received per city was $2,514. For Milwaukee, the largest city in the State, it amounted to $109,700, based on 548.5 miles of streets not included in the State highway system. Eight of the cities received over $10,000 each. Forty-seven cities received between $1,000 and $10,000, and 86 received less than $1,000 each. The sum of this amount of $404,824, which was distributed among the cities and the $37,540 given to villages for municipal street improvements, or a total of $442,365, constituted but 3.4 per cent of net revenues derived from motor-vehicle fees and the gasoline tax.

North Carolina allows any city or town to charge a license fee not exceeding $50 for any motor vehicle used in transporting persons or property for hire in lieu of all other charges, fees, and licenses.

Oklahoma laws provide that from the 60 per cent of funds raised by the motor-vehicle tax and returned to counties from which collected, the board of county commissioners are required to pay to the incorporated towns and cities of their counties 15 per cent of all money collected in such towns and cities to be credited to the street and alley fund.

The laws of Washington transfer out of the motor-vehicle fund the sum of $1,400,000 annually to the credit of the primary highway maintenance fund from which latter fund is transferred annually to each first and second class city a sum equal to $500 per mile for each mile of primary highway in such city, and similarly to each third and fourth class city $300 per mile.

Ohio cities receive a part of the license fees and also of the gasoline tax receipts. Fifty per cent of the license fees is turned back to the municipalities in which the tax was collected. Thirty per cent of the gasoline tax receipts is also returned to the municipalities. The gasoline tax apportionment is made on the basis of the number of vehicles registered. In 1925, although the gasoline tax was in effect only after April 18, the apportionments of revenues from this source amounted to $2,702,985. That year the total license fee revenues were $13,147,231, and one-half of this amount was transferred to counties and municipalities. Just what portion of this transfer of funds went to municipalities is not disclosed by the reported figures. The funds derived from the license fees and the gasoline tax are to be used by cities for street maintenance purposes only.

In New York the law provides that 25 per cent of the license fees collected in each county shall be retained by the county for road maintenance purposes to be expended under the direction of the State commissioner of highways. Inasmuch as the city of New York comprises several counties its share of the license-fee revenues is credited to the city's general fund, the law being silent as to how the city is to use these funds.

The analysis of the statutory provisions relating to the disposition of the motor-vehicle fees and gasoline taxes by the States shows that where the States are directed to transfer a portion of these funds to counties or municipalities a number of different bases are used in making the apportionments. In 22 States there is such a transfer of funds. In 16 States the amount of fees collected from local government units, such as county or city, is named as the basis of apportionment; in two States it is the number of registered motor vehicles. One State uses population as a basis; one State uses area, one State uses road or street mileage; and one State makes an equal distribution among the counties.

There are 21 States which distribute a portion of the gasoline tax receipts. Five States distribute these receipts in proportion to the amounts collected; 4 distribute them according to mileage; 3 use the number of motor vehicles registered as a basis; and 3 divide the portion to be distributed equally among the counties. One State uses population as a basis of distribution; one uses the amount of State taxes paid as a basis. Area and mileage, population and area, mileage and equal distribution among counties, registration and equal distribution among counties furnish the bases in four other States, respectively.

The total disbursements in 1925 for rural highway purposes are reported to have been $1,288,939,707. This amount includes the outlays for highway improvements made by the State highway commissions and by the local authorities. In arriving at this total there are included as expenditures the transfers of funds from State commissions to local authorities and also the transfer from local to State authorities. After making a deduction of these transfers of funds the net total disbursements are found to be $1,241,049,094. The total collections from motor vehicles, including license fees and gasoline taxes, amounted to $406,648,561, an amount equivalent to 32.8 per cent of the net disbursements. Not all of these motor-vehicle revenues which the States collected were used or were available for highway purposes.

The net disbursements made by the State highway commissions were $623,549,605 in 1925. To meet these expenditures the States received $199,845,163, or 32 per cent of the disbursements, from motor-vehicle license fees; and $89,328,340, or 14 per cent, from gasoline taxes. It may, therefore, be said that $289,173,503, or 46 per cent, of the net highway disbursements made on behalf of State highway commissions was derived from motor-vehicle revenues.

The net disbursements for local highways were $617,499,489. To meet these expenditures the local highway authorities received $71,379,424, or 11.5 per cent, of the total disbursements as their share of the motor-vehicle revenues. The sum of $46,545,445, or 7.5 per cent, of the disbursements was derived from motor-vehicle license fees, and $24,833,979, or 4 per cent, came from gasoline taxes.

These funds for highway purposes derived from motor-vehicle license fees and gasoline taxes and their relation to the total net disbursements, State and local, for highway improvements may be summarized as follows:

Total net highway disbursements, State		$623, 549, 605
Total net highway disbursements, local		617, 499, 489
Total		1, 241, 049, 094
Sources of highway funds, State:		
License fees	$199, 845, 163	
Gasoline tax	89, 328, 340	
Total		289, 173, 503
Ratio of these sources to State highway disbursements, 46 per cent.		
Sources of highway funds, local:		
License fees	$46, 545, 445	
Gasoline tax	24, 833, 979	
Total		71, 379, 424
Ratio of these sources to local highway disbursements, 11.5 per cent.		
Total funds derived from motor-vehicle license fees and gasoline tax, State and local		360, 552, 927
Ratio to total highway disbursements, State and local, 29 per cent.		

The disbursements covered the items of construction on roads and bridges, maintenance, administration and engineering, retirement and payments on bonds, interest payments, equipment, machinery and repairs, etc. The funds to meet these expenditures in addition to those derived from motor-vehicle revenues were obtained from bond issues, taxation, and Federal aid. The Federal-aid funds may also be regarded as having been derived from the excise taxes on automobiles collected by the Federal Government inasmuch as the

(Continued on p. 229)

EARTH PRESSURES ON CULVERT PIPES[1]

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STRUCTURES that are constructed of elastic materials having practically uniform physical properties, and which are subjected to applied loads of known magnitude, direction, and position may be safely designed in accordance with an exact mathematical theory. This is not true, however, of structures which are subjected to applied loads caused by the weight of granular materials, because the loads become variable depending upon the movement of the granular materials and, in consequence thereof, the attendant variable frictional and cohesive forces. In other words, the deflection or deformation of the supporting structure is, in this case, an important function governing the magnitude and direction of the applied loads. Therefore, in order to determine the effect of loads on structures due to the action of granular materials, such, for instance, as earth embankments on culvert pipe, it becomes necessary to perform large-size experiments that will conform as far as possible to actual practical conditions.

Certain experiments on a large scale extending over a period of several years with the object in view of solving this problem have been conducted by Dean Anson Marston, of the Iowa State College at Ames, Iowa.

The apparatus at Ames was apparently designed on the assumption of obtaining earth pressures on a rigid body practically free from deflection or deformation. The earth pressure of embankments, varying from 16 to 20 feet in height, acting on wooden cylindrical bodies 40 inches in diameter and supported from the top of the inside, were transferred by a system of levers to a number of platform scales where the readings were observed and recorded.

In the second progress report presented April 7, 1922, at Chicago to the joint concrete culvert pipe committee, Dean Marston stated that his experiments showed that the maximum pressures obtained on the cylindrical body as determined from actual scale readings for a projection condition was about 1.9 times the weight of the material directly over the cylinder.

Other experiments to determine pressures on culvert pipe have been made near Farina, Ill., under the auspices of a subcommittee of the American Railway Engineering Association, and directed by J. R. Wilks, former chief engineer of the Armco Culvert and Flume Manufacturers' Association.

The embankment from which these pressures were obtained was 35 feet high and located on a new line of the Illinois Central Railway. Different kinds of pipes were used, consisting of corrugated metal, heavy cast iron, and reinforced concrete, with sizes ranging from 24 inches to 42 inches in diameter. The apparatus used to obtain the pressures consisted of a series of soil pressure cells, commonly known as Goldbeck cells, which were attached to the inside surface of the pipe. The results of these experiments as found in the report of the subcommittee of the American Railway Engineering Association are as follows:

> The intensity of vertical pressure measured by the top cells of the corrugated or so-called flexible culvert is about 54 per cent of the weight of the unit column of material above the cell, whereas for the so-called rigid type of culvert the cell reading showed 1.58 times the weight of the material above the cell. Or, in other words, the intensity of vertical pressure measured by the top cells on the flexible type is about one-third of the measured vertical pressure on the rigid type of culvert.

In 1922, Frank Page, chairman of the North Carolina Highway Commission, and Charles M. Upham, chief engineer, made the suggestion to the staff of the engineering school of the University of North Carolina that a series of experiments be undertaken at Chapel Hill to determine earth pressures on culvert pipe, using conditions that would conform to the practice as closely as possible. Accordingly, a site was chosen for the experiments on a private roadway near the Pittsboro Road, about 1 mile south of Chapel Hill, N. C. The conditions surrounding the site are admirable for the performance of the experiments as it is accessible to the main highway with sufficient working material close at hand. The profile of the original ground permitted a good location for the culvert pipe apparatus and the formation of embankment up to 20 feet in height, or greater if desired.

Test Pipe in Place

DESIGN OF APPARATUS

The main objects to be observed in the design (fig. 1) were to eliminate, as far as possible, the movement of the supporting element under the culvert pipe and provide an ample place for the weighing apparatus and convenient space for the observer.

The housing for the weighing apparatus consists of a reinforced concrete box culvert 58.5 feet long, 7 feet wide, and having a clear height of 7 feet. The ground for the foundation of the culvert box was excavated sufficiently at the lowest point of the roadway so as to obtain the desired elevation of the test pipes.

The weighing apparatus which was placed inside of the culvert box consists of four platform scales, each having a capacity of 30,000 pounds, designed and constructed especially for these experiments by Riehle Bros., of Philadelphia. They have been calibrated a number of times and are considered to give quite accurate results. Steel columns extending up to the roof of the culvert box are supported on the scale platform and on top of these columns the test pipes are placed.

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[1] This paper by Dean Braune and the accompanying discussion by Dean Marston were presented at the annual meeting of the American Association of State Highway Officials, at Pinehurst, N. C., Nov. 12, 1926. The investigations reported in Dean Braune's paper were made in cooperation with the North Carolina State Highway Commission and the U. S. Bureau of Public Roads. The work was directly supervised by Prof. H. F. Janda, of the engineering school of the University of North Carolina. The author desires to acknowledge his indebtedness to Dr. William Cain for valuable advice and assistance, and to the manufacturers by whom the pipe used in the experiments were furnished.

The columns are only 5 feet long but in order to eliminate any appreciable deformation they were provided with an excessive cross-sectional area so as to keep the unit stress below 1,000 pounds per square inch. The concrete roof of the culvert box is supplied with a slot large enough to permit the insertion of the test pipes. In the first series of experiments the four test pipes, consisting of cast iron, were 30 inches in diameter and 2½ feet long, with a barrel thickness of 1 inch. These pipes were submerged in the roof of the culvert so that one-half of the pipe extended above the top surface of the roof. This condition would be termed a "50 per cent projection."

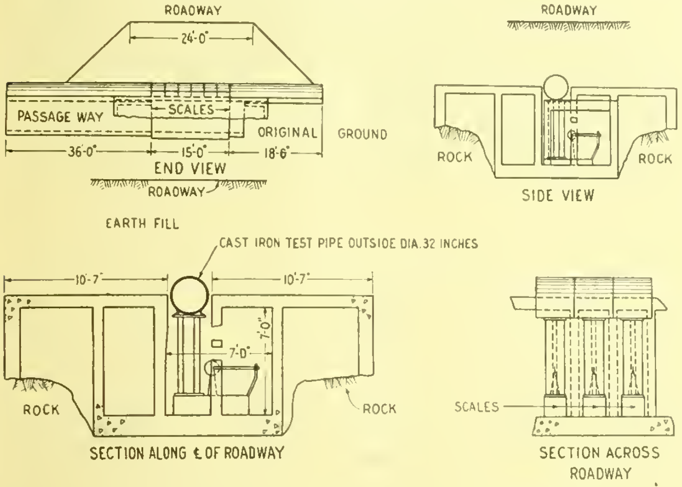


Fig. 1.—Sketch of Apparatus for Earth Pressure Experiments at Chapel Hill, N. C.

In the second series of experiments which are now under way, the bottom of the pipe is placed flush with the top surface of the culvert roof, constituting what is termed a "100 per cent projection."

The filling material consisted of sand of a fairly uniform character, obtained from a small creek about a half mile from the site. Besides the sand, a fill material of clay was used in one set of experiments.

The embankment was given a minimum width of 24 feet at the top when the fill had reached its maximum height of 20 feet with side slopes of 1½ to 1.

Concerning the action of earth pressures on elastic culvert pipes the following is quoted from a paper entitled: "Design of Culvert Pipes," by Dr. William Cain:

> The simple laws of mechanics and certain experiments indicate that the vertical earth pressure, *E*, on pipe culverts varies according to the relative deflections of the top of the pipe and the adjacent soil. In diagram No. 1 (fig. 2) the top of the pipe deflects much more than the adjacent soil and there will be a tendency for the prism directly over the pipe to move downward, relative to the earth alongside it, and through friction and cohesion it transfers part of its weight to the soil adjacent to the pipe. Therefore, the earth pressure, *E*, is less than *W*, the weight of the material directly over the pipe. In the case illustrated by diagram No. 2, the deflections of the top of the pipe and the soil adjacent are assumed to be the same and there will be equal settlement of both the prism of earth and the adjacent soil, no relative motion ensuing. Consequently, there will be no friction exerted along the sides of the prism and the earth pressure, *E*, will then be equal to *W*. The other case, illustrated by diagram No. 3, is just the converse to that of the first diagram. The earth outside the prism over the pipe is moving downward, relatively to the prism itself, this being due to the rigidity of the pipe. Thus, through the medium of friction and cohesion, part of the weight of the outside material is transferred to the prism over the pipe and *E* becomes much greater than the weight of the prism *W*.

We will call the ratio of the actual earth pressure to the weight of the prism of earth directly over the pipe, *K*—that is, $K = \frac{E}{W}$. Now, in accordance with the theory as expressed above, *K* should vary, depending on the relative deflections of the pipe and the earth alongside of the pipe, or in other words its value would be dependent on the rigidity of the pipe. For flexible pipe, therefore, *K* should be less than unity, but greater than unity for the rigid type of pipe.

The Weighing Chamber

FIRST TESTS MADE WITH PIPE IN 50 PER CENT PROJECTION

Four sections of cast-iron pipe, each having a diameter of 30 inches, a length of 2½ feet, and a barrel thickness of 1 inch, were placed on the weighing machines so as to permit a 50 per cent projection—that is, 50 per cent of the circumferential surface of the test pipe extended above the concrete roof surface of the box culvert. (The concrete roof surface here corresponds to the original ground surface alongside of pipe.) The embankment was made of sand carefully deposited on top of and on each side of the pipe for a distance of 12 feet.

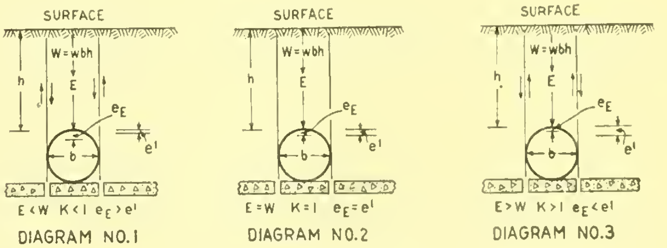


Fig. 2.—Diagrams Illustrating the Action of Earth Pressures on Elastic Culvert Pipes, According to Dr. William. Cain

The embankment was formed in 1-foot layers permitting the side slopes to assume the natural repose.

During the construction of the embankment, scale readings were taken at 1-foot intervals and at the same intervals samples were taken for the determination of the physical properties of the sand. In order to record active earth pressures the deflection of the surface upon which the earth bears should be in the same direction as the pressure. Therefore the tendency of the movement of the scale platform should be in a downward direction. This condition was obtained by running the rider out so that the scale beam was always down and the platform up. When the readings were taken the rider was run in until the beam just began to float. Thus the mass of earth tended to move down and was supported in part by friction and cohesion. The vertical earth pressure exerted upon the pipe was read on the scales.

Except during actual work on the fill, or during pressure observations, a tarpaulin was kept over the surface in order to maintain the moisture content as constant as possible.

A preliminary test was first made in which the embankment was carried up to an elevation of 7 feet. The passive pressures were obtained for this test so as to get a comparison with the active ones. After removing this fill a new one was started on June 11, 1924, in which the active pressures were obtained and recorded for each 1-foot interval until a height of 20 feet was reached on October 1, 1924. The results of this experiment are shown in Figure 3, in which is recorded the scale pressure curve and also a curve showing the actual weights of the prism of material resting on top of the pipe.

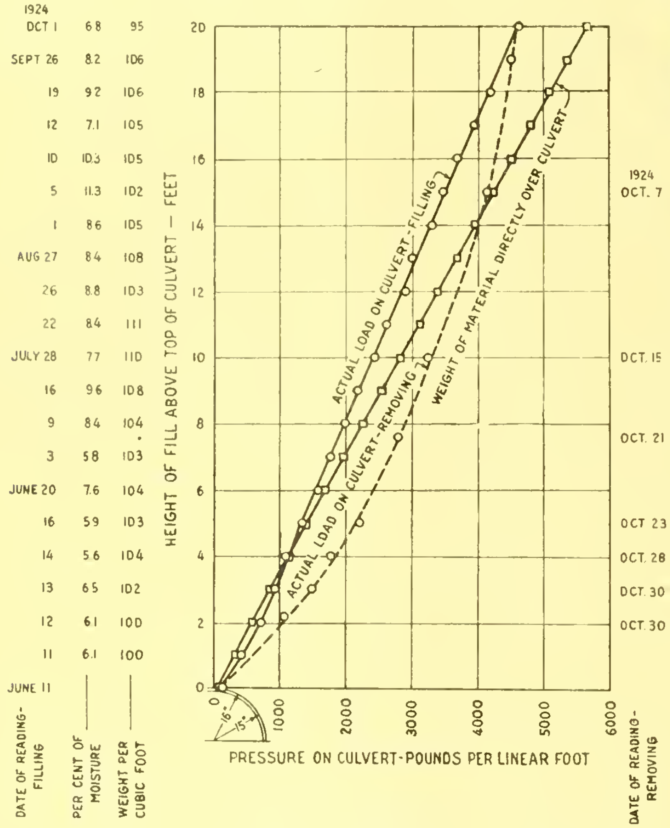
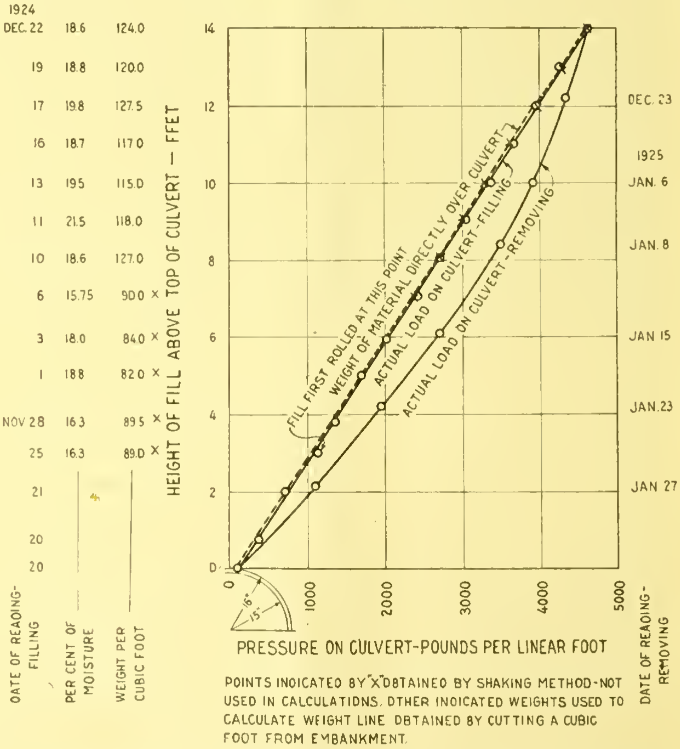


FIG. 3.—RESULTS OF FIRST EXPERIMENT AT CHAPEL HILL, N. C., SHOWING PRESSURES ON CULVERT CORRESPONDING TO VARIOUS HEIGHTS OF SAND FILL DURING THE PLACING AND REMOVAL OF THE FILL. THE TEST WAS MADE ON A CULVERT WITH 50 PER CENT PROJECTION

If we call *W* the weight of the prism of material immediately over the test pipe, and *E*, the pressure transmitted as observed from the scale readings we obtain for this experiment a ratio $K=\frac{E}{W}=0.84$. In other words, this experiment showed that the earth pressure, *E*, on the test pipe is 84 per cent of the weight of the earth prism immediately over the test pipe. This ratio remains practically constant for the entire height of the fill except for the first 4 feet, where the earth pressures are greater than the weight of the earth prism.

FILL AT CHAPEL HILL, N. C., AT END OF TIME AND WEATHER TEST

It should be noted that this ratio, *K*, as given above, was obtained for a 50 per cent projection of the pipe. In the second series of experiments that are now being performed the pipe has been placed so as to obtain a 100 per cent projection—that is, the sides of the pipe will be fully surrounded by filling material. Under this method of laying the pipe, a greater deflection of the material alongside the pipe should be obtained, and consequently in accordance with the theory as outlined above, *K* should be greater than for the 50 per cent projection.



POINTS INDICATED BY "X" OBTAINED BY SHAKING METHOD-NOT USED IN CALCULATIONS. OTHER INDICATED WEIGHTS USED TO CALCULATE WEIGHT LINE OBTAINED BY CUTTING A CUBIC FOOT FROM EMBANKMENT.

FIG. 4.—RESULTS OF THE TEST AT CHAPEL HILL, N. C., SHOWING PRESSURES ON A CULVERT, IN 50 PER CENT PROJECTION, CORRESPONDING TO VARIOUS HEIGHTS OF CLAY FILL, DURING THE PLACING AND REMOVAL OF THE FILL

Referring again to the first sand fill, the fill having been built up to a height of 20 feet, was then removed by stages and readings were taken at intervals during its removal. It was observed that for equal heights the pressures during removal were greater than during the filling, and these pressures are shown in comparison with those recorded at the same heights during filling in Figure 3.

CLAY FILL AND 50 PER CENT PROJECTION

The next test to be undertaken was that of a 50 per cent projection condition and clay fill. The procedure in obtaining the scale pressures was the same as in the case of the sand filling, except that the clay was rolled at each 1-foot layer after the third with a 4-ton road roller. The 14-foot clay fill was started on November 20 and completed December 22, 1924, and the results of the test are recorded in Figure 4 where it is seen that the pressures are about equal to the weight of the prism over the pipe—i. e., the ratio $K=1$.

The clay fill was then removed and the next test undertaken was that of the trench-condition clay fill. The trench condition as suggested by Dean Marston designates that method of laying in which the pipe is laid in a trench so that the top of the pipe is below the ground surface alongside. It was believed that under this condition the pressures would be lessened, and this theory was borne out by the test.

With the test pipes in place a clay fill was made up to a height of 4.15 feet above the roof of the culvert. A trench was then dug across the entire width of the fill down to the concrete slab so that the test pipe sections were uncovered. The original width of the trench was 43 inches and the depth 48 inches to the top of the pipe. After the trench had been dug, it was filled in as loosely as possible with the clay which had been removed from it and the forming of the embankment was then resumed. From this point the fill was constructed in the same manner as the projection clay fill and was completed to a height of 11 feet when a layer of sand 6 inches thick was spread on the surface and the fill was allowed to remain in place from February 25, 1925, to May, 1926.

The pressures for this trench condition were found upon completion of the embankment to be less than for the projection condition, the ratio (K) of the pressure to weight of the prism over the pipe decreasing, as the fill was raised, from a value of unity at the 4-foot level, to approximately 0.6 for the 10-foot height (fig. 5).

The pressures, read at frequent intervals during the year that the embankment was in place, decreased from March 3, 1925, to the middle of August of the same year, when a total of 33 per cent decrease had taken place. After that time a slow increase occurred and at the last reading, May 3, 1926, a third of the lost pressure had been recovered.

Sudden variations in the weather had but little effect upon the pressure, but it can be seen from the curve, Figure 6, that a heavy rainfall produces a slight temporary increase.

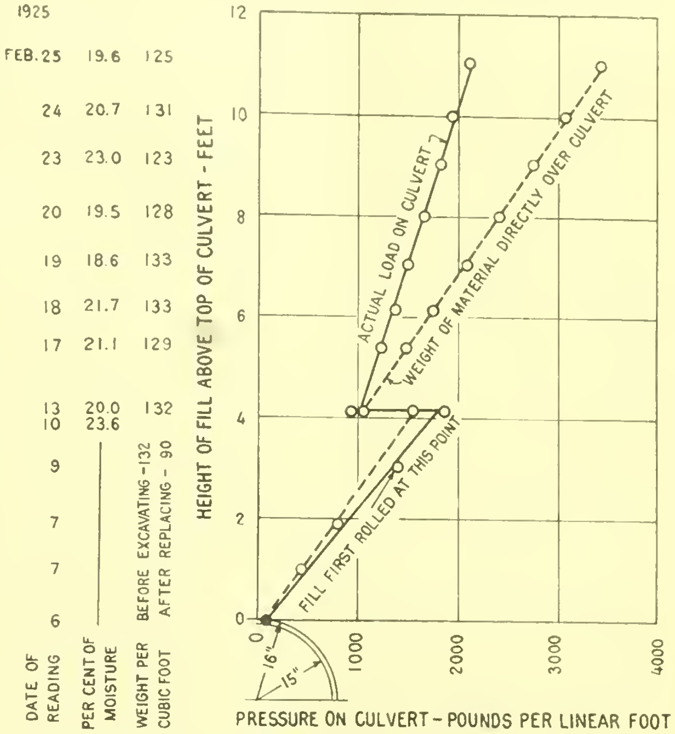


Fig. 5.—Results of Tests of Pipe Laid Under a Trench Condition in Clay Fill Showing Pressures on Culvert Corresponding to Various Heights of Fill

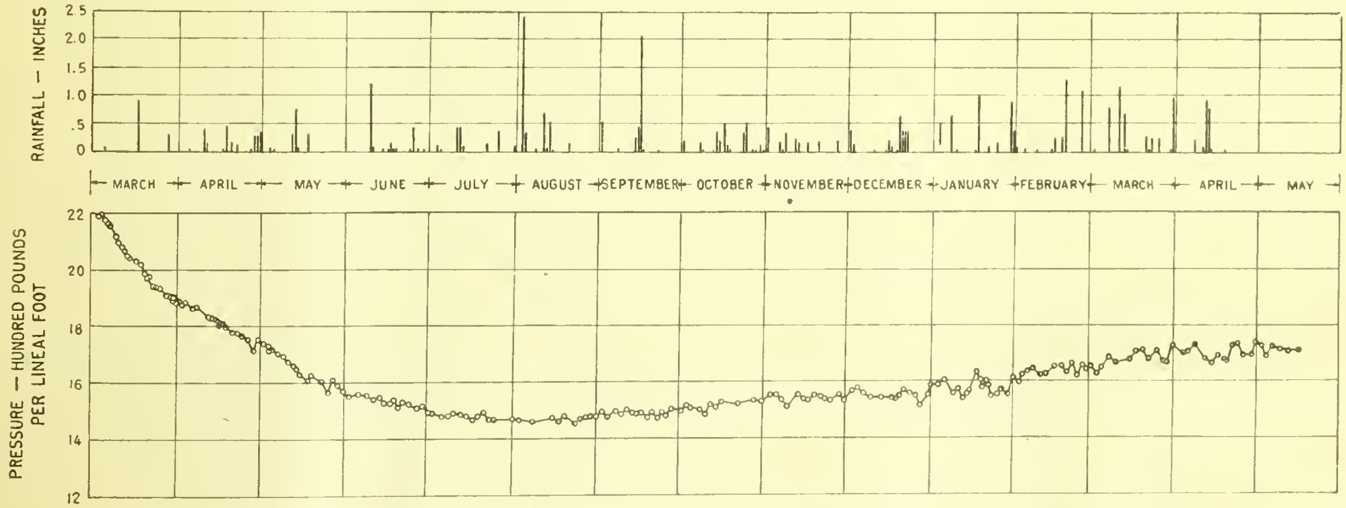


Fig. 6.—Effect of Time and Weather on the 11.05-Foot Trench Condition Clay Fill

100 PER CENT PROJECTION IN SECOND TEST SERIES

The second series of experiments was undertaken with the idea of ascertaining the influence of deflection of the pipe on the pressures. It was believed that the pressure was an important function of the deformation or deflection of the culvert pipe. It was therefore decided that a series of experiments should be performed, using test pipes of different flexibility, varying from a practically rigid body to one of large deflection, but using the same apparatus that had previously been employed.

It was also proposed to use pipes 30 and 20 inches in diameter in order to obtain a relation of pressures for different sizes. In this series of experiments, however, the bottoms of the pipe are placed flush with the floor of the culvert, thus establishing a condition of 100 per cent projection.

Five different kinds of pipes of varying flexibility having a diameter of 30 inches and a length of 2 feet 6 inches have so far been tested. These consist of a solid concrete plug of practically no deformation, cast-iron pipe with a barrel thickness of 1 inch, smooth steel pipe five-sixteenths inch thick, corrugated pipe No. 12 gauge, and a smooth wrought-iron pipe of No. 12 gauge.

The fill material so far has consisted of the same kind of sand that was used in the first series of experiments, deposited in the same manner as heretofore described.

Four pieces of each kind of pipe were placed on the weighing machines with the bottom of the test pipe flush with the top surface of the culvert roof.

Embankments 12 feet in height have been built for each set of experiments, and scale readings have been observed and recorded for each 1-foot interval. In addition to the scale readings it was deemed desirable to obtain pressures from soil-pressure cells and accordingly each of the four test pipes for the different sets of experiments were supplied with seven pressure cells.

When the experiment is completed and the large amount of data obtained have been thoroughly studied and digested, an attempt will be made to find a correlation between the experimental data and the practical design for the strength of culvert pipe. However, the tests so far performed confirm the theory on culvert pipes, namely, that the pressures vary inversely as some power of the deflection of the pipe, and values of the ratio, K, have been obtained ranging between limits above and below unity.

DISCUSSION

By **ANSON MARSTON**, Director, Iowa Engineering Experiment Station

The Iowa Engineering Experiment Station has been engaged in research to determine the loads supported by underground conduits (of which pipe culverts are one class) for 17 years. For several years this work has been carried on in cooperation with the United States Bureau of Public Roads; and the station expects to continue the work actively for several years.

Five bulletins and one progress report have already been published at various dates from 1913 to 1926;[1] and other reports will be published in cooperation with the Federal bureau in the near future.

The Iowa culvert researches have resulted in a complete mathematical theory of vertical loads and active horizontal pressures on culverts of all sizes, shapes, and degrees of flexibility, which apparently is well borne out by extensive experimental and field data obtained by the station, and which is easily fitted to Dean Braune's experiments and to the tests conducted by the corrugated pipe interests for the American Railway Engineering Association, at Farina, Ill., as published by the association early this year and as referred to by Dean Braune.

Culvert pipe in common use are of two types: (1) Rigid culvert pipe, made of burnt clay, plain concrete, reinforced concrete, or cast iron; and (2) flexible culvert pipe, made of corrugated sheets of iron or steel.

Flexible culvert pipe.—The flexible pipe are not of themselves able to carry much vertical load, but under usual conditions of use they distort sufficiently to greatly lessen the vertical load by shortening their vertical diameters and, by materially lengthening their horizontal diameters during distortion under the vertical load, they develop important passive horizontal resistances in the surrounding materials, sufficient to bring about a condition of equilibrium somewhat changeable, it is true, but usually pretty stable.

Rigid culvert pipe.—The rigid pipe, on the other hand, do not distort enough, up to the point of objectionable cracking and distortion, to greatly reduce the vertical load or to develop passive horizontal resistances which can safely be counted upon to help materially. Only active horizontal pressures, such as are usually calculated by Rankine's formula (say, one-fourth to one-third the vertical pressures in the materials alongside), can safely be counted upon to help sustain the vertical load.

Our mathematical theory of loads on culverts enables the computation of the vertical loads on both rigid and flexible culvert pipe (and on rectangular culverts as well) for all conditions of construction, and for different yielding of foundations as well, provided that the relative shortening of the vertical height of culvert, settlement of the culvert foundation under load, settlement of the granular filling materials alongside, and settlement of the original surface alongside under the embankment weight can be forecast approximately.

Already data are available from which safe loads (sometimes perhaps larger than are sure to develop) can be calculated.

FINE CONDITIONS OF INSTALLATION RECOGNIZED

For load calculation purposes, culvert construction conditions, for both pipe and rectangular culverts, can be divided into five classes, illustrated by Figures 1 to 5, inclusive.

1. *Ditch condition.*—This is the loading condition in which all the filling material in the prism above the culvert moves down relative to the materials alongside as it settles. The typical case is that in which the culvert is constructed in a ditch reaching to the top of the embankment, but there are several other cases which bring about the ditch condition. The load on the cul-

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[1] Buls. 31, 36, 47, 57, and 76, and Second Progress Report on Culvert Pipe Investigations, Iowa Eng. Exper. Sta.

vert under this condition is given by the formula—

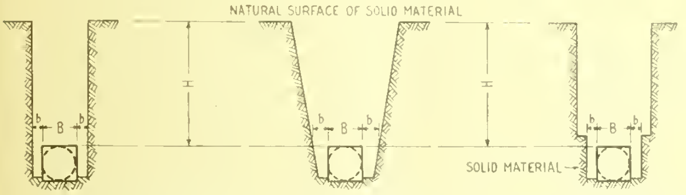
$$W = C_d w\ (B+2b)^2 \qquad (1)$$

where W = load on the culvert in pounds per linear foot
w = weight of material in pounds per cubic foot
B = breadth of culvert in feet
b = clearance between the side of the culvert and the side of the ditch in feet

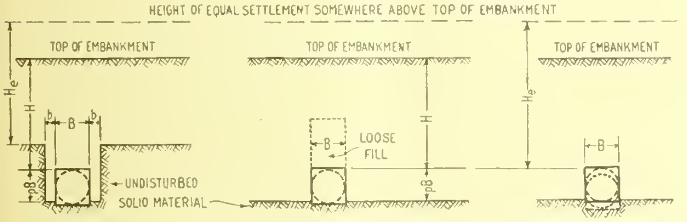
and $$C_d = \frac{1 - \dfrac{1}{\epsilon^{2K\mu\frac{H}{B}}}}{2K\mu} = \text{ditch coefficient}$$

in which ϵ = the base of Naperian logarithms
μ = the coefficient of friction of ditch filling against the sides of the ditch.
H = height of ditch filling above top of pipe in feet.
K = the ratio of lateral to vertical earth pressure.

Formula (1) is now widely established in engineering literature.[2]

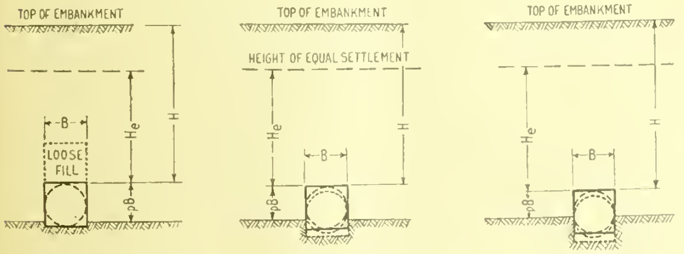


(a) Conduit in trench with vertical sides (b) Conduit in trench with steep, sloping sides (c) Conduit in narrow trench in solid material at bottom of wider trench


HEIGHT OF EQUAL SETTLEMENT SOMEWHERE ABOVE TOP OF EMBANKMENT
TOP OF EMBANKMENT
He
H
b
B
pB
UNDISTURBED
SOLID MATERIAL
LOOSE
FILL

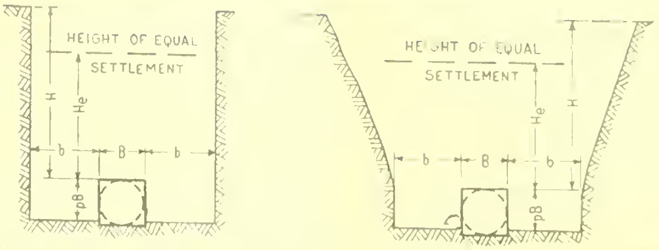
(d) Conduit in trench in solid materials with embankment overhead (e) Sufficient loose fill over conduit (in 1 or more layers) to insure greater settlement of all material over conduit than of materials alongside (f) Yielding of foundation plus shortening of vertical diameter sufficient to insure greater settlement of all material over conduit than of materials alongside

FIG. 1.—TYPICAL EXAMPLES OF DITCH CONDITION OF LOADS ON CONDUITS FROM TRENCH FILLS OR EMBANKMENT MATERIALS



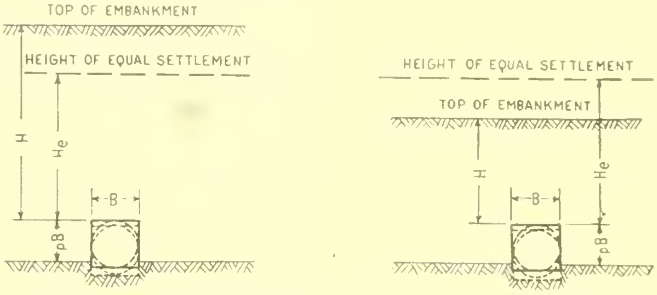
(a) Loose fill sufficient to insure greater settlement of material over conduit than of materials alongside only to a level below top of embankment (b) Yielding of foundation and shortening of vertical diameter sufficient to insure greater settlement of material over conduit than of materials alongside only to a level below top of embankment (c) Yielding of foundation and shortening of vertical diameter sufficient to insure greater settlement of material over conduit than of materials alongside only to a level below top of embankment

FIG. 2.—TYPICAL EXAMPLES OF IMPERFECT DITCH CONDITION OF LOADS ON CONDUITS FROM TRENCH FILLS AND EMBANKMENT MATERIALS



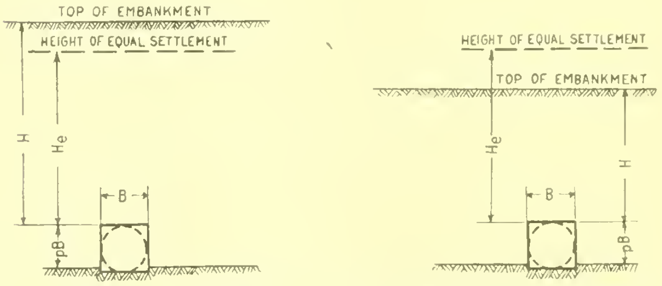
(a) Conduit in trench with vertical sides (b large in proportion to pB). (b) Same as (a) except sides of trench sloping

FIG. 3.—TYPICAL EXAMPLES OF THE COMBINED DITCH-PROJECTION CONDITION



(a) Yielding of foundation plus shortening of vertical diameter considerable, but less than settlement of material alongside originally at level of top of conduit (b) Yielding of foundation plus shortening of vertical diameter considerable, but less than settlement of material alongside originally at level of top of conduit. Same as (a) except that height of equal settlement is above top of embankment

FIG. 4.—TYPICAL EXAMPLES OF THE IMPERFECT PROJECTION CONDITION



(a) Foundation rock or other quite unyielding material. Conduit rigid pipe or rectangular in shape (b) Same as (a) except that height of equal settlement is above top of embankment

FIG. 5.—TYPICAL EXAMPLES OF THE PROJECTION CONDITION

2. *Imperfect ditch condition.*—In this the ditch condition is established imperfectly by loosely filled material over the pipe, or by shortening of the vertical diameter or by yielding of the foundation or by a combination of these circumstances. Formula (1) usually applies part way up, after which the load increases at a rate equal to R_1wB, in which R_1 is a constant greater than unity. In another case, however, the load equals wB up to a height of equal settlement, H_e, above which the load increases at a rate R_2wB, in which R_2 is a constant less than unity.

3. *Combined ditch and projection condition.*—This condition is encountered when a culvert is built projecting above the bottom of a ditch which is relatively very much wider than the outside breadth of the culvert. The projection condition formula (see 5) applies up to a height of equal settlement, H_e, above which the load increases according to the ditch-condition formula (see 1).

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[2] See Bul. 31, Iowa Engineering Experiment Station, 1913.

4. *Imperfect projection condition.*—In this the culvert projects above the original surface alongside, on which the embankment rests, but there is a material shortening of the vertical diameter under load, or settlement of foundation, or both, sufficient to lighten the vertical load on the culvert, yet less than the settlement of the embankment materials alongside at the level of the top of the culvert. The load is given by the projection-condition formula (2) slightly modified (see 5).

5. *Projection condition.*—In this the culvert projects above the original surface alongside and is built on foundations which do not settle materially relative to the original surface alongside, on which the embankment is built, and the vertical height of the culvert is not shortened materially by the load.

The load for the projection condition is given by the formula [3]

$$W = C_p w B^2 \text{-----------------------------------} (2)$$

$$C_p = \frac{\epsilon^{2K\mu\frac{H_e}{B}} - 1}{2K\mu} + \left(\frac{H}{B} - \frac{H_e}{B}\right)\epsilon^{2K\mu\frac{H_e}{B}} = \text{projection coefficient}$$

Figure 6, from the second progress report, 1922, provides the means by which formulas (1) and (2) may be simply applied to calculations of loads on culverts.

Figures 7 (Bul. 31), 8 (second progress report), and 9 (1926 A. R. E. A. Proc.) show conclusively that formulas (1) and (2) are confirmed closely by actual culvert load measurements.

The original tests reported by Dean Braune fall between formulas (1) and (2), except his trench-condition clay fill, which checks closely with formula (2) to a height of 4 feet and with formula (1) above 4 feet.

HEIGHT OF EQUAL SETTLEMENT MEASURED

The mathematical derivation of formula (2) demonstrates the theoretical existence in the fill of a height of equal settlement, H_e, at which the materials over the pipe and those alongside settle equally. Unpublished data obtained by the Iowa Engineering Experiment Station in 1925–26, demonstrate by actual measurements the existence of such a height of equal settlement, approximately at the computed level.

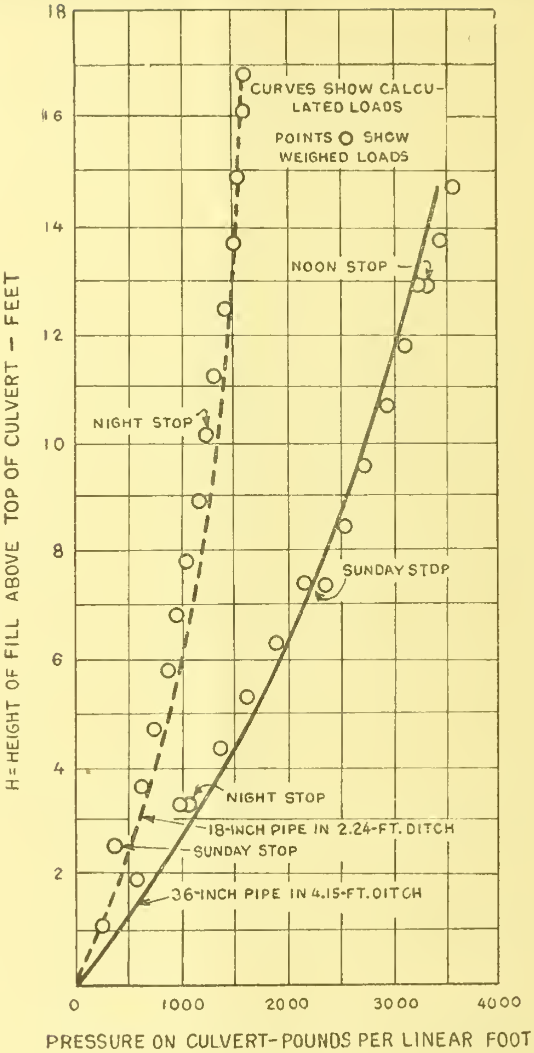

Fig. 7.—Comparison of Calculated Loads on Pipes with Actual Weight of Loads

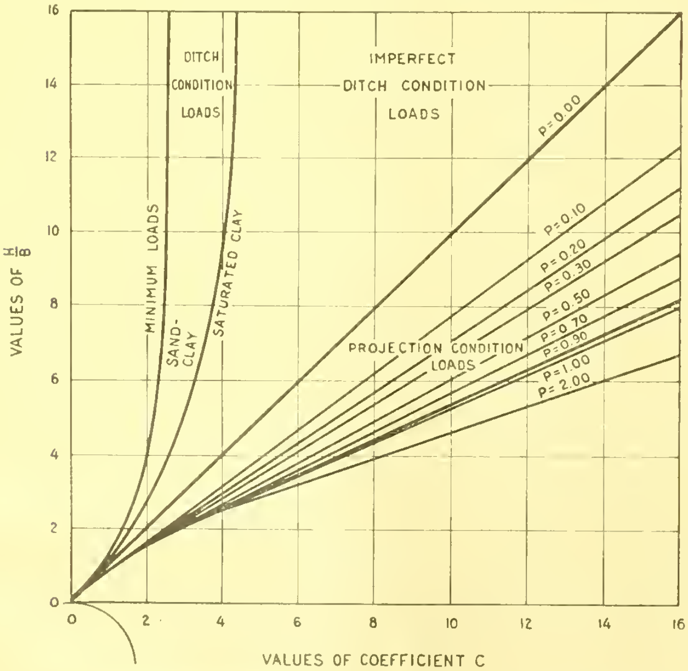

Fig. 6.—Computation Diagram for Loads on Culverts

Many measurements reported in Bulletin 76, Iowa Engineering Experiment Station, 1926, show that for 36-inch concrete culvert pipe, the shortening of the vertical diameter under load up to the point of cracking is insignificant relative to the settlement of the foundation and materials alongside. The Farina experiments show that the same was true for 42-inch cast-iron pipe, the vertical diameter of which shortened only 0.3 inch under 35 feet of fill, whereas the culvert settled 3½ inches into the foundation.

This discussion has referred so far to loads on pipes due to filling materials only. R. W. Crum's report on field examinations of culverts shows that the great majority of culverts are built under fills of 5 feet or less. In all such cases the load on the pipe due to the embankment is nearly equal to the actual weight of the materials directly over the pipe. But, in such shallow fills, important additional superloads are transmitted to culverts as a result of the weight and impact of vehicles.

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[3] See Second Progress Report on Culvert Investigations, Iowa Engineering Experiment Station, April, 1922.

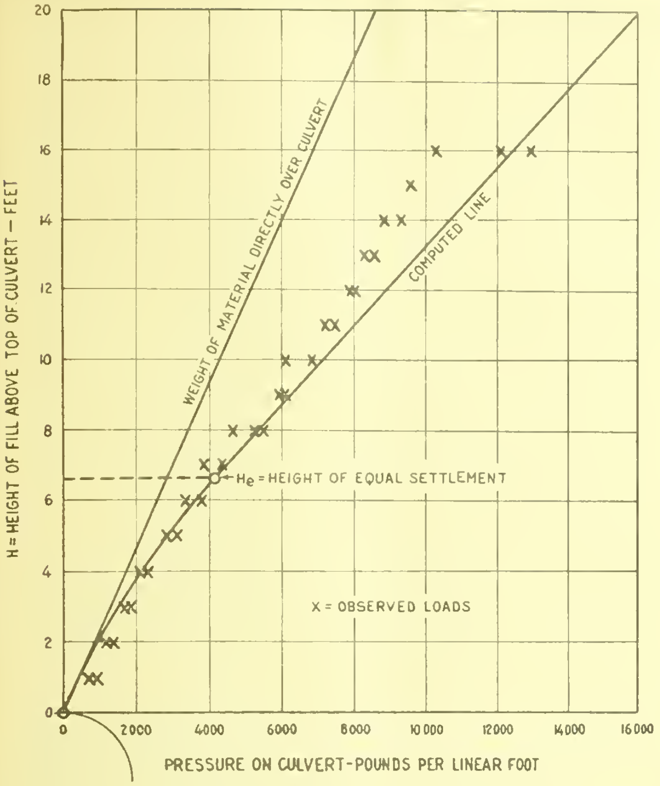

Fig. 8.—Comparison of Actual and Theoretical Loads on Pipe. The Material of the Fill was Sand and Gravel with a Weight of 126.5 Pounds Per Cubic Foot

The Iowa Engineering Experiment Station will shortly publish a bulletin in cooperation with the Bureau of Public Roads giving the results of three years active experimentation with loads transmitted to culvert tops from static and moving superloads. These show that culverts should be designed to carry superloads equal to

$$W_s = IC_tT \text{ - } (3)$$

Where W_s = total allowance for superload and impact

I = impact coefficient = 1.50 to 2.00

T = heaviest weight on one wheel

$$C_t = a\sum \frac{3}{2\pi}\frac{H^3}{H_s^5}$$

In the formula for C_t a is one of the small uniform unit areas into which the top of the culvert must be divided for summation calculations. H is the vertical distance from top of culvert to the roadway as used before and H_s is the slant height from center of unit area to point of application of superload to roadway. The expression $\frac{3}{2\pi}\frac{H^3}{H_s^5}$ would if multipled by the superload give the intensity of vertical pressure at the center of the small unit area corresponding to the value of H_s.

In conclusion, it is desired to call the attention of highway engineers to the great differences in the actual supporting strength of culvert pipe which may be caused by variations in the bedding of the pipe during construction. The several conditions of bedding classified and the effect of each upon the safe load are as follows:

Impermissible bedding, in which the pipe is laid without bedding below the horizontal diameter, may weaken the culvert pipe 20 per cent or more.

Ordinary bedding, refers to the case in which the bottom of the pipe is carefully bedded for 90 per cent of the circumference.

First-class bedding, in which, in addition to ordinary bedding, the earth is very carefully deposited and tamped under and around the pipe, may increase the strength 25 per cent.

And, finally, properly designed concrete cradle bedding, may double the supporting strength. A forthcoming bulletin of the Iowa Engineering Experiment Station, 1926 (the Bureau of Public Roads cooperating), gives the results of tests of such beddings with reinforced concrete pipe up to 84 inches in diameter.

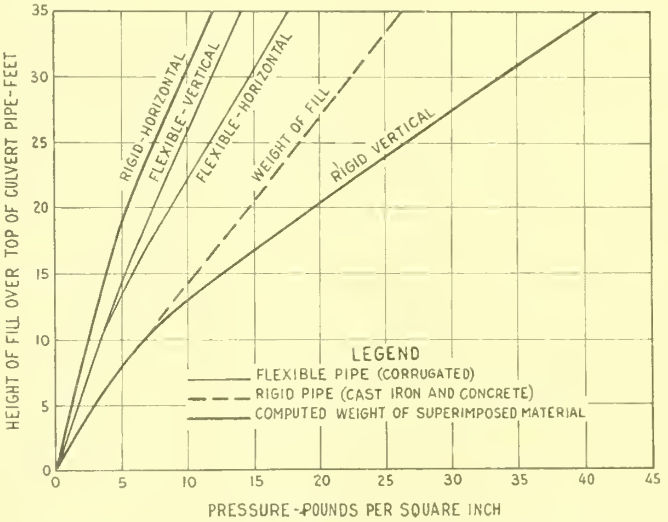

Fig. 9.—Approximate Average Vertical Pressures on the Top of Rigid and Flexible Culvert Pipes Corresponding to Various Heights of Fill as Shown by the Farina Tests (Bul. 284, Vol. 27. 1926, A. R. E. A.).

(Continued from p. 221)

Federal-aid payments for highway work have been up to this time very much less than the total amount collected through this tax since the tax has been in effect. The Federal-aid payments in 1925 amounted to $95,749,998, which is equivalent to about 7.7 per cent of the country's total rural highway disbursements. If these figures are added to the funds derived from motor-vehicle revenues used for highway purposes, we arrive at an amount equivalent to 36.7 per cent of the total disbursements. Although a larger sum was collected in the form of license fees and gasoline taxes, the funds derived from these sources actually used for highway purposes were considerably less. Before any further attempts are made to withdraw a still larger part of the motor-vehicle receipts and use them for purposes other than for rural highways, a careful study should be made as to the extent to which highway expenditures in the various States are met through motor-vehicle contributions. A part of these motor-vehicle revenues is also used for administrative expenses. It behooves each State in that connection to see to it that such costs are kept at a minimum so that as large a share of the license fees and gasoline taxes as possible shall be devoted to the purpose for which they are intended.

THE DOWNWARD KICK OF THE REAR WHEELS OF VEHICLES IN STARTING FROM REST TO MOTION

By THOMAS K. A. HENDRICK, Highway Research Board, National Research Council

When a vehicle, at rest on the highway, starts, what is the measure of the effect on the pavement of the changed condition of rest to motion? The configuration, mass, and acceleration of the vehicle, obviously, determine the answer. Doubtless the answer would be given qualitatively, for it is believed that no quantitative measurements have ever been made. Qualitatively one would say that the pavement does not suffer in the slightest. It is not known that any highway specifications account for this specific force. But it is conceivable that the increase in the size and weight of motor trucks and the increasing number of quick-starting taxicabs, may make a quantitative answer desirable.

Experiments have, therefore, been made by the writer, along lines suggested by Prof. C. J. Tilden, chairman of the department of engineering mechanics, Yale University, on a bicycle, 3 Buick cars, 3 motor trucks (5 to 13 tons gross weight), and 5 taxicabs. The measurements were made by means of Toledo scales whereon the increase of force can be read directly from the dial scale as the indicator advances.

For the bicycle, the 550-pound scale, in the Sheffield laboratory of engineering mechanics, Yale University, was used. Through the courtesy of the Connecticut State Highway Department, the 40,000-pound scales located at West Haven, Conn., were used for automobiles, trucks, and taxicabs. All measurements were made with the rear wheels only on the platform of the scales.

When a vehicle is started from rest, according to D'Alembert's principle of reversed effective forces, there is applied at the center of gravity of the mass accelerated, as a result of the inertia of the vehicle, a reversed effective force equal to the mass times the acceleration and opposite in direction to the direction of motion. This reversed effective force, when added to the system of external forces, produces an imaginary condition of static equilibrium in all the forces acting on the vehicle. For this condition the laws of static equilibrium, $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$, rigorously hold.

Applying the principle of the summation of moments about the point of contact of the front wheel, as a point of moments, it is readily seen that the reversed effective force causes a downward thrust or downward "kick" on the rear wheels, and an uplift on the front wheels. This downward "kick" was registered in the experiments made on the dial of the scale by the rapid advance of the indicator. The difference in the reading before and during starting measures the downward "kick" in pounds.

It is important to emphasize that this registration of the downward "kick" in pounds on the dial of the scale is independent of the acceleration and the location of the center of gravity in so far as it is not computed from these quantities. It is, in itself, a direct measurement of the downward "kick."

RESULTS OF EXPERIMENTS

For each of the vehicles experimented with, 20 readings were made except in the case of the bicycle for which 21 readings were made. Each reading represents a separate, independent start, and the average of the readings was taken as the "kick" caused by each vehicle. The start was a normal start except in the case of the Buick 6-cylinder car. By "normal start" is meant that the vehicle was started as one would normally start on getting into a car on the highway and driving off. The drivers were experienced, their experience ranging from one to five years. The gasoline was controlled by foot accelerator except in the case of the Buick car, above mentioned, in which it was controlled by the throttle which was opened to the position necessary for a speed of 35 miles per hour on a level road.

As a result of the tests, it is believed that it can be stated that the downward "kick" of the rear wheels of vehicles, in starting from rest, varies according to the type and classification of the vehicle from 4 to 23 per cent of the load on the rear axle; and from 2 to 11.8 per cent of the total load.

SHOWING THE SCALES USED FOR MEASURING THE DOWNWARD KICK OF AUTOMOBILES AND TRUCKS. FOR NORMAL STARTS THE AVERAGE DOWNWARD KICK OF THE 4-CYLINDER, 5-PASSENGER TOURING CAR SHOWN WAS 7.6 PER CENT OF THE LOAD ON THE REAR AXLE

For light passenger vehicles, with about 1,700 pounds on the rear axle and 3,200 pounds total weight, the average downward "kick" is about 6.4 per cent of the load on the rear axle or about 3.4 per cent of the total load.

For trucks with a total weight of 22,000 pounds 13,000 pounds on the rear axle, the downward "kick" is about 5.3 per cent of the load on the rear axle or 3.2 per cent of the total load.

For taxicabs, with a total weight of 3,500 pounds, and a weight on the rear axle amounting to 1,900 pounds, the downward "kick" is about 18.2 per cent of the latter or about 9.1 per cent of the total load.

In some cases, depending upon the load and body characteristics of the vehicle, and the idiosyncrasies of the driver, the downward "kick" of the rear wheels of some passenger cars may easily be one-fourth of a ton and for heavy trucks it may easily be one-half a ton.

MOTOR TRUCK IMPACT TESTS NOW IN PROGRESS

A special series of field motor-truck impact tests, now being carried on by the Bureau of Public Roads, is expected to be completed within a few months. The objectives of these tests were determined from a study of the data secured during the original program which has been completed and reported on.[1] The work is in cooperation with the Society of Automotive Engineers and the Rubber Association of America.

Based upon the findings of the original program, it is believed that motor-truck impact reactions are dedendent on four major variables, namely, road roughness, tire equipment, wheel load, and vehicle speed. It is not believed that the effect of one variable should be considered without due regard for the other three. From the original program it was possible to plot lines of equal impact reaction for one road condition, four tire types, and various combinations of load and speed. The result was a series of isodynamic curves from which could be read the maximum total vertical reactions in thousands of pounds that might be reasonably expected to occur on that particular road for any combination of wheel load and truck speed. The data were segregated by tire types such as pneumatic, new cushion, new solid, and worn-out solid equipment; and the curves represented a range of wheel loads up to 20,000 pounds and speeds from 0 to 30 miles an hour. The tests were made on the Arlington test road which had been roughened by artificial obstructions.

With this preliminary isogram as a basis, a program of tests was formulated, the field work of which is expected to be completed within a few months. A number of highway sections have been selected as representative in type and roughness. These sections were marked off in one-twentieth mile lengths and points were spotted on the pavement with traffic white to guide the trucks over the test sections. The road sections were calibrated carefully at varying speeds by means of the "roughometer" described in the September, 1926, number of PUBLIC ROADS. The roughness limits ranged from 100 to 800 units at a speed of 30 miles per hour.

Tire equipment was selected to represent the four types used in the preliminary investigation, namely, pneumatic, new cushion, new solid, and worn-out solid. The average deflections of the four types under a static load of 10,000 pounds were approximately 2.5 inches, 1 inch, 0.7 inch, and 0.2 inch, respectively. The tires were mounted permanently on extra wheels in order that tire changes might be made rapidly and conveniently by substituting wheels.

At the beginning of the tests the rear-wheel loads were standardized at 2,500, 5,000, 7,500, and 10,000 pounds. Two trucks were used, a 2-ton truck for the 2,500 and 5,000 pound loads and a 5-ton truck for the other loads. The light load of each truck was accurately measured on platform scales and securely fastened in position. Then the heavier load was built up on each truck by adding 100-pound lead or iron weights. The positions of these extra weights were marked on the trucks so that the wheel loads might be duplicated readily.

The trucks were operated at speeds varying by small increments from the minmum up to the maximum speed obtainable. The average speed of each run was computed from stop-watch observations and the known length of the test section. Variations in speed were observed from speedometers mounted on the trucks. Runs varying from the average speed by more than one-half mile per hour as registered by the speedometer were not recorded.

A field-test program of this character made necessary a mobile organization and equipment. To provide for this a field office and two loading and storage platforms were built on a 16-foot, 3-ton trailer chassis. On a 2-ton truck chassis a special platform body was constructed with a swinging chain hoist placed at the forward end. In addition to hauling the trailer this service truck was used to carry the spare wheels, gasoline, and general equipment necessary for the tests.

The 2-ton and 5-ton test trucks were equipped with the coil-spring accelerometers developed by the bureau, which were mounted so as to follow the vertical motion of the right rear wheels.

Municipal authorities in the District of Columbia and in Alexandria, Va., were interested cooperators in the tests made within their jurisdiction. Traffic officers were assigned to assist in the work, and they were extremely valuable in making it possible to conduct the tests on the streets with safety. No parking signs were placed temporarily at several points to facilitate the tests.

The value sought in these tests for each combination of the four variables was the average of the five highest impacts occurring within the one-twentieth mile section. This represents the magnitude of the average impact which occurs at the rate of 100 per mile. Since the unsprung component of motor-truck impact is generally the important quantity, it was considered sufficient to compute this unsprung component and add to it the sprung component determined by static weighing in order to arrive at the total impact force. An attempt will also be made to express the absolute maximum in terms of the average of the five highest impacts. The indications at present are that useful and reasonably accurate results may be obtained.

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[1] See PUBLIC ROADS, vol. 7, No. 4.

Apportionment of Federal-aid funds for the fiscal year ending June 30, 1928

State	Amount
Alabama	$1, 547, 220
Arizona	1, 056, 994
Arkansas	1, 277, 896
California	2, 483, 437
Colorado	1, 376, 520
Connecticut	472, 685
Delaware	365, 625
Florida	899, 451
Georgia	1, 979, 209
Idaho	935, 193
Illinois	3, 154, 429
Indiana	1, 926, 772
Iowa	2, 044, 999
Kansas	2, 068, 532
Kentucky	1, 417, 947
Louisiana	1, 013, 308
Maine	680, 794
Maryland	635, 119
Massachusetts	1, 089, 100
Michigan	2, 214, 691
Minnesota	2, 120, 741
Mississippi	1, 307, 879
Missouri	2, 405, 175
Montana	1, 551, 499
Nebraska	1, 585, 138
Nevada	948, 510
New Hampshire	$365, 625
New Jersey	934, 611
New Mexico	1, 186, 763
New York	3, 635, 217
North Carolina	1, 713, 356
North Dakota	1, 194, 951
Ohio	2, 762, 209
Oklahoma	1, 751, 891
Oregon	1, 182, 202
Pennsylvania	3, 335, 735
Rhode Island	365, 625
South Carolina	1, 054, 988
South Dakota	1, 220, 064
Tennessee	1, 614, 766
Texas	4, 497, 272
Utah	846, 906
Vermont	365, 625
Virginia	1, 442, 714
Washington	1, 131, 532
West Virginia	793, 636
Wisconsin	1, 870, 455
Wyoming	934, 369
Hawaii	365, 625
Total	73, 125, 000

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
*1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94C. TNT as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60M. Federal Legislation Providing for Federal Aid in Highway Construction.
62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.

FARMERS' BULLETINS

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*505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

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*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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*Department supply exhausted

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

DECEMBER 31, 1926

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,725,985.09	1,298.3	$ 1,708,147.90	$ 825,708.63	101.0	$ 5,498,744.76	$ 2,593,686.42	287.9	$ 295,817.78	$ 147,908.89	22.7	$ 3,603,385.97	Alabama
Arizona	10,949,878.25	5,863,772.36	729.8	521,097.34	352,011.23	35.7	1,343,390.41	919,184.58	75.9	22,957.56	16,607.49		3,522,667.35	Arizona
Arkansas	18,364,544.50	7,666,698.35	1,323.0	1,360,033.39	667,684.72	96.6	3,963,716.51	1,859,735.54	254.1	1,443,663.80	720,968.79	98.4	1,968,612.60	Arkansas
California	27,142,596.90	13,003,592.30	1,058.0	4,037,071.01	2,045,065.55	149.7	9,688,765.12	4,682,948.28	208.2	64,943.48	29,364.45	1.7	4,795,281.42	California
Colorado	13,905,904.64	7,127,288.18	745.0	958,040.35	522,177.14	37.9	5,857,348.44	2,856,362.73	266.8	211,960.04	118,880.34	17.1	3,077,623.61	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1	415,786.78	153,899.62	8.0	5,998,783.30	1,672,924.81	76.1				878,955.77	Connecticut
Delaware	4,918,052.29	1,781,665.60	124.3	741,382.64	306,931.14	17.6	915,210.80	373,034.05	25.4	194,011.77	10,000.00	7.1	368,052.21	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	2,120,466.67	1,049,833.65	62.7	9,007,138.20	4,040,758.40	229.1	318,826.56	125,724.10	6.5	1,943,726.53	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	2,962,273.28	1,453,529.53	193.0	10,706,286.40	5,253,001.01	468.8	15,264.89	7,632.44	4.4	2,032,761.16	Georgia
Idaho	11,061,198.14	5,882,112.70	724.7	1,708,277.16	929,954.24	79.0	2,128,711.33	1,335,307.38	169.9	375,382.97	229,622.70	30.3	1,117,822.98	Idaho
Illinois	44,116,611.86	20,619,996.74	1,377.7	2,012,307.84	980,960.66	68.1	10,137,571.69	4,866,489.69	361.6	665,850.50	325,680.26	24.1	6,193,490.65	Illinois
Indiana	16,949,425.87	8,172,125.19	534.3	2,537,876.26	1,199,358.66	76.0	17,337,352.74	8,157,219.31	497.3				2,602,423.84	Indiana
Iowa	29,062,375.40	11,926,302.10	2,114.8	3,272,974.60	1,491,965.64	212.0	12,412,418.41	5,434,917.20	639.3	1,557,438.01	381,288.35	56.6	2,296,088.71	Iowa
Kansas	32,826,601.64	12,590,489.25	1,160.6	678,298.12	303,264.40	55.5	14,324,663.10	5,659,543.16	800.2	2,401,379.15	837,355.37	93.2	2,142,290.82	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	1,277,442.23	486,957.94	48.9	7,074,068.96	3,352,830.21	348.8	1,326,319.76	656,264.63	62.6	1,642,620.97	Kentucky
Louisiana	13,830,592.68	6,144,739.99	1,055.9	886,239.31	393,861.22	31.9	3,779,484.19	1,821,118.75	189.7	1,318,275,09	608,819.21	40.6	1,317,176.83	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6	644,409.76	271,914.49	21.9	3,417,586.53	1,198,470.65	95.3	138,832,05	69,416.02	5.7	1,413,313.45	Maine
Maryland	10,924,943.10	5,112,991.22	423.3	193,172.47	96,586.23	15.7	1,560,297.86	703,957.44	72.7				646,641.11	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.5	412,120.06	121,949.75	5.1	5,468,749.89	1,483,354.92	78.2	1,085,305.70	308,090.10	20.7	2,627,170.61	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	854,634.64	404,909.66	28.4	14,222,085.99	6,454,713.62	406.5	856,335.00	363,026.50	21.5	3,507,353.92	Michigan
Minnesota	37,170,985.95	15,586,116.56	3,161.9	7,860,229.10	3,430,029.11	461.6	2,756,734.28	494,800.00	202.2	3,445,644.84	48,000.00	163.4	2,153,575.33	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1,129.0	1,131,550.92	563,148.80	71.7	7,939,100.95	3,872,016.84	408.8	128,702.10	64,351.05	13.0	1,521,846.21	Mississippi
Missouri	28,989,166.92	13,736,014.85	1,543.2	6,857,542.11	3,018,747.35	214.3	14,385,915.26	5,702,559.25	382.9	2,124,659.26	937,679.73	101.5	1,796,609.82	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	1,156,333-27	752,162.86	65.5	1,721,275.20	1,471,555.90	143.3	875,848.89	462,880.20	94.5	5,956,319.15	Montana
Nebraska	11,533,401.62	6,474,202.52	1,768.3	1,922,511.90	948,935.00	224.8	13,064,579.56	6,378,254.32	1,359.6	494,339.17	235,345.62	46.0	3,183,635.54	Nebraska
Nevada	7,558,195.51	5,130,934.59	538.8	2,190,619.83	1,879,898.84	215.1	1,784,241.70	1,547,311.92	226.5				1,185,579.65	Nevada
New Hampshire	4,992,658.60	2,377,450.07	237.6	104,262.46	50,101.07	3.6	1,443,736.05	655,765.31	41.4				451,800.55	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	5,575,268.35	2,302,312.27	19.7	4,617,894.57	907,168.56	58.4	232,146.38	42,765.00	2.9	1,051,442.96	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0	36,773.39	21,877.50	0.6	2,968,018.56	2,035,691.50	249.5	300,577.16	184,067.12	9.9	2,577,855.50	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	2,501,509.66	934,544.57	58.1	40,669,973.00	10,572,982.70	665.2	5,189,000.00	1,229,797.50	67.0	7,031,130.04	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9	5,677,359.00	2,274,655.41	124.7	4,380,616.71	2,025,475.48	135.6	497,429.00	238,440.59	20.1	1,714,652.58	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	2,588,498.16	1,258,555.51	355.1	5,535,261.05	2,965,233.20	807.9	532,299.12	266,150.00	79.2	1,421,811.51	North Dakota
Ohio	47,699,532.90	17,371,787.03	1,364.1	2,771,725.59	1,160,121.47	100.9	12,857,841.20	4,876,402.19	374.6	1,408,279.96	557,593.66	17.9	4,528,100.65	Ohio
Oklahoma	28,247,950.33	13,159,999.15	1,178.9	973,143.32	475,218.56	41.7	3,474,535.94	1,408,119.84	197.3	1,993,265.31	682,396.34	78.6	2,085,944.11	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	702,784.65	403,640.92	23.7	3,250,886.94	1,816,407.15	128.0	345,457.42	7,000.00	11.8	1,241,286.14	Oregon
Pennsylvania	61,366,160.80	21,560,732.04	1,168.8	250,508.96	47,888.62	3.2	29,858,176.82	8,526,177.50	597.2	3,083,860.09	1,093,440.41	64.9	3,446,277.43	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	274,781.19	113,520.00	7.6	2,026,527.69	531,795.00	35.4	239,058.07	74,175.00	5.0	754,874.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1,481.9	1,471,111.01	571,994.39	71.8	5,540,232.74	2,374,083.28	190.2	99,046.24	42,921.58	10.3	1,101,189.82	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,181.2	1,045,179.92	517,228.44	177.9	3,748,415.33	1,901,410.05	655.0	113,029.83	30,539.05	12.9	1,333,849.49	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	1,199,774.63	567,186.82	44.3	9,011,110.36	3,867,387.29	246.7	822,734.62	279,030.92	24.7	1,905,167.95	Tennessee
Texas	69,163,673.48	27,440,254.72	4,920.2	4,385,425.62	1,997,934.76	278.3	17,586,290.20	7,861,596.06	744.6	2,726,243.68	1,302,431.02	82.2	6,501,486.45	Texas
Utah	8,253,178.03	5,098,440.68	546.4	825,999.14	615,614.05	79.1	1,510,499.60	1,143,262.36	124.4	708,494.96	539,986.25	40.6	1,268,381.66	Utah
Vermont	4,242,042.64	2,017,699.51	134.5	144,445.28	66,281.32	3.2	2,152,080.83	826,574.77	39.9				723,576.40	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.5	1,238,406.67	553,255.39	40.8	5,886,599.53	2,524,847.71	164.9	1,006,321.00	30,640.23	24.1	1,449,756.66	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	300,577.74	116,642.49	8.7	4,382,234.09	2,045,600.00	95.0	140,581.59	52,000.00	7.0	1,280,156.05	Washington
West Virginia	9,473,716.44	4,141,062.65	392.9	951,130.88	432,685.36	26.5	5,634,425.23	2,201,847.66	164.3	1,434,758.96	529,950,92	84.5	840,600.41	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1	101,729.42	50,664.71	9.3	8,937,409.85	4,264,448.88	399.0	64,661.52	32,330,00	0.2	4,569,120.68	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.5	878,558.47	559,909.00	107.6	2,485,341.14	1,577,526.63	178.6				1,322,320.32	Wyoming
Hawaii				343,664.15	97,440.00	6.5	1,277,417.35	393,912.64	16.4				974,425.36	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	84,763,456.60	39,840,718.69	4,190.6	365,729,746.36	151,489,782.13	14,580.6	40,298,997.28	13,920,561.83	1,575.4	113,070,233.77	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 125,978,145.08 Federal aid $ 51,768,588.79 Miles 4,534.4

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 12 ▽ FEBRUARY, 1927

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ROAD CONSTRUCTION OVER PEAT BOGS HAS PRESENTED A DIFFICULT PROBLEM IN MICHIGAN

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FILL SETTLEMENT IN PEAT MARSHES[1]

By V. R. BURTON, Engineer on Special Assignments, Michigan State Highway Department

THE MICHIGAN State Highway Department has just completed a special study of fill settlement in peat marshes as a part of the general soil survey it has undertaken. Settlement in peat marshes has caused serious trouble in many places, because it was unknown how much fill material would be required and even after the fill had been completed there was no assurance that it would maintain the original grade line. It was therefore desired to ascertain the physical characteristics of the peat and decide on the field data necessary to properly locate and estimate the cost of a road over a peat deposit. It was hoped that it would be possible to devise a method of construction more economical than that then in use, and to adopt better maintenance methods to restore sunken fills and prevent further settlement.

A brief description of peat deposits will be necessary if any adequate idea is to be had of what actually exists under the surface of a "sink hole." Peat deposits are classified and mapped stratigraphically in essentially the same way as mineral upland soils. For general peat studies, peat materials have been described by layers in terms of composition, texture, structure, and color. This gives a fairly good idea of the main differences between separate peat layers and localizes their relative positions.

VARYING CHARACTERISTICS OF PEAT LAYERS AID STUDY OF FILL SETTLEMENT

The classification of peat strata was made in accordance with the system largely developed in this country by Dr. Alfred P. Dachnowski, of the Bureau of Plant Industry, United States Department of Agriculture, and he personally supervised a number of surveys. The following description of peat deposits is taken from an article on the "Stratigraphic Study of Peat Deposits"[2] by Doctor Dachnowski. "Distinction should early be made between muck and peat. Muck is the thin surface layer of disintegrated peat and is entirely different in appearance and properties to the relatively unaltered peat below it. This distinction is made because many engineers use the terms interchangeably with no knowledge that there is a difference in the two."

Table 1 is from the article just referred to, and gives the classification of characteristic peat layering.

Pulpy peat is formed in water basins under conditions of poor drainage from transported organic sediment carried from its origin and redistributed by water currents. There are no visible bedding planes although the original sediment was laid down in practically horizontal position under conditions differing by rate of deposition, seasonal changes, etc. The plant remains are small in size. In texture the peats vary from coarse to very finely divided particles, while in structure a layer may vary widely in its degree of compaction. Fresh from the deposit and while still moist the colors vary from gray, green, and brown to black.

DRAINAGE DITCH THROUGH PEAT BOG

As with most peats, the pulpy variety quickly loses its character by oxidation on exposure to the air and drying. It shrinks enormously and becomes a hard, water-resisting substance when quickly dried out. It will not then absorb anything like the original quantity of water. Doctor Dachnowski states that "it may be accepted as an axiom that undrained deposits of peat contain from about 70 to 95 per cent of water."

The outstanding characteristic of fibrous layers of peat is their matted or felt-like, porous nature which has its origin in the slightly altered remains of moss, roots, rootlets, and rhizomes of herbs. The peat layers differ widely in character. Their water content is enormously high, the solid matter commonly composing only from 5 to 15 per cent of the peat mass. In texture they vary from coarse to very finely fibered material usually presenting a rather loose, porous appearance, with colors ranging from gray through yellow to red brown and dark brown. The layer may be interbedded or it may grade into a layer of woody of pulpy peat by admixture or it may occur in overlapping beds of other kinds of fibrous peat.

TABLE 1.—*Structural units of peat deposits and some of their physical characteristics*

Groups of peat-forming vegetation	Types of peat	Character of peat layers	Color of peat layers	Texture of peat layers	Structure of peat layers
Aquatic	Macerated	Pulpy (sedimentary)	Olive green, brown to black.	Coarse to very finely divided.	Compact, impervious, stiff, plastic or loose, friable.
	Colloidal				
Marsh	Reed	Fibrous	Gray, red or yellow brown to dark brown.	Coarse to fine fibered	Dense, matted, felty, or porous, spongy.
	Sedge				
	Brown moss				
Bog	Bog moss				
	Heath shrub	Woody	Dark brown to blackish brown.	Coarsely fragmented to granular.	Compact and granular, or loose, wicker-like.
Swamp	Willow-alder shrub				
	Deciduous forest				
	Coniferous forest				

---

[1] Paper presented at the fifth annual meeting of the Highway Research Council, Washington D. C., December, 1926.
[2] Soil Science, vol. 17, No. 2, February, 1924.

Woody peat may occur either as the dominant component or as a prominent admixture in other types. The woody plant remains are broken down partly into granular débris and partly into irregularly shaped woody fragments, which are almost unaltered by decomposition. The woody peats differ widely in composition and character. They may be as finely divided as sawdust or as coarse as a tangle of waterlogged brush and logs. Several layers may be found as, like the fibrous peats, they are developed on moist flat land under conditions of a rising or fluctuating water table. Land may grow up to forest, be submerged to form a marsh, and then emerge again to become a forest. In one peat deposit examined on this survey three distinct forest layers were plainly visible.

Marly phases of peat are formed from calcareous plant remains and shells of mollusks and are usually found admixed in the macerated types of peat. Extensive deposits of marl with peat occur mainly in regions underlain by limestone rock or where the soil adjacent to and underneath a peat area is derived from limestone drift. Deposits vary in color from gray to brown and are usually soft plastic masses with particles from fine to comparatively coarse, depending on the amount and character of plant or shell remains present.

The lake clay found at the bottom of most deep peat deposits is structureless, mainly siliceous material from plant and animal remains consisting of diatomaceous shells and drift débris. It is gray to gray blue in color, fairly compact, but with a high percentage of water in it and is quite plastic and sticky. It is, however, a much more compact and weighty material than any of the peats or marls.

In carrying out the study of fill settlement it was decided to cross section marshes accurately by borings, which would show the various peat layerings and the exact position of the filled material with reference to them. Samples of the various classes of peat were to be collected and sent to the laboratory for physical tests in an effort to discover some relationship between their bearing power and the known settlement for various water contents. If a large number of existing fills were thus examined, it seemed logical that general conclusions might be drawn to serve as a guide on future work.

FILL SETTLEMENT NOT AFFECTED BY CHARACTER OF PEAT

Fills over eight different peat deposits were cross sectioned at 83 different points where the depth of the peat ranged from 1 to 66 feet. The shape of the fill material was accurately determined with a 2-inch Empire drill. Cores from the peat were taken with the Davis peat sampler where possible, but if the peat was compacted beyond a certain amount the Empire drill and jet was used. Enough test holes were put down on the cross section chosen to show the fill shape fairly accurately and an effort was made to extend the cross section in both directions from the fill sufficiently far to get out of the zone where the layers were disturbed by filling. In a few cases it later became evident that the test holes at the extremities of the cross section were not placed far enough from the center line.

Samples of the different peat layers were collected at various depths and sent to the laboratory for physical tests. Results were very disappointing in this regard. It was impossible to maintain the peat in its original condition on exposure to the air. Peat as it comes from the ground is only partially oxidized and at the greater depths there is actually some reducing action. Consequently, immediately it is handled in air, oxidation begins and if it is attempted to dry it, a totally different material results. Laboratory bearing value tests were not satisfactory. It is certain that the amount of compression under loading in a vessel such as is used in the standard bearing value test which does not permit of a free displacement in all directions will not give results at all indicative of the behavior of the same material in its natural place.

The only way, then, to secure the solution of the various problems involved in this study was by a careful study of the cross sections showing the fill shape and the peat layerings in the disturbed position after filling. An effort was made to reconstruct the position of the layering as it originally existed from the position of the layers outside the distorted area of the cross section. A study of the fill cross section and the distortion of the original layers as reconstructed led to the conclusions drawn from these effects.

The various factors which it was thought should determine the amount of settlement were depth and character of peat layering, height of the water table, slope of the mineral subsoil beneath the peat, height and width of fill. The weight and character of the fill material would also be a factor but the difference in weight of the materials studied was so small as to be neglected. All fills in this investigation were earth and varied from a clay to sand. No rock fills were studied.

So far as could be determined the character of the peat layering is very slightly responsible for differences in fill settlement. There is, to be sure, a decided difference between the top layer of peat a foot or two in depth where the material is fairly well decomposed, but this is due more to a smaller water content than in the actual character of the material itself. The lake clay which occurs in the bottom of most of our deeper peat deposits is enough different in character to make it necessary to consider its effect separately. It is also probable that if any considerable thickness of pure marl with a heavy texture were encountered it should be grouped with the clay or given separate treatment. As it happened in this investigation no very great amounts of marl were found except as they occurred as a peat admixture. A soft soupy marl should probably be classed as peat in considering its effect on fill settlement. The preliminary work of sounding a peat bog preparatory to estimating fill settlement is thus considerably simplified for the engineer, as where a uniform marl is absent, only two layers, the peat and lake clay, need be considered. No differentiation of peat layering is necessary for estimating fill settlement.

DEPTH OF PEAT AND LAKE CLAY THE MOST IMPORTANT FACTORS

If there were any great variation in the height of the water table compared to the depth of the deposit, this factor should certainly show its effect. It so happened that the road ditches were about the same depth through all of the shallow peat deposits studied and in the deposits of greater depth the depth of the ditch was very small in comparison to the total depth of the peat. The water table in most cases varied from 1 to 3 feet below the fill surface. The varying height of water table, which was to a certain extent dependent on the time of year as well as on the particular situation of the cross section, was not taken into account. Some inconsistencies in the shallower depths are probably due to the neglect of this factor.

The height of the fill above marsh surface is of course a factor, although not so great a one in the case of loading over a wide area as one would expect. In peat depths of less than 10 feet the settlement is small and is affected by factors other than height of fill and results are inconsistent. The height of fill produces a marked effect in medium depths of peat of from 10 to 25 feet. In depths beyond 25 feet the total depth of fill is so great in proportion to the height above marsh surface that the amount of this additional loading is not important. The above refers to completed fills studied and not to new construction methods in which the height of fill placed in the early stages is of considerable importance.

The slope of the mineral subsoil produces a marked effect on settlement and the bottom of the fill nearly always takes the same direction of slope as the marsh bottom. This is due to two reasons; first, the flow of peat beneath the fill is aided by gravity, and, second, the greater depth of peat down slope induces greater settlement. Most of the inconsistencies in amount of settlement from the determined normal occur above sharp slopes in the mineral subsoil. The amounts varied so much, however, that no general law could be observed. It is safe to say that in general if the mineral subsoil does slope quite sharply, the normal amount of settlement will be increased.

One very important factor met in special cases is the effect on fill settlement of the peat compression by neighboring fills. The compressed peat is much more resistant to flow than is the uncompressed peat, and hence flow on the side adjacent to an old fill is very much smaller than on the free side.

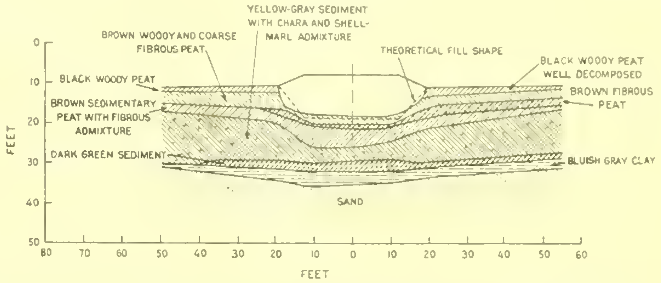
The factors of greatest importance and the ones least difficult of determination are simply the depth of the peat layer and the depth of the lake clay beneath it. Other factors undoubtedly have their influence but the depths alone, so far as could be determined from the information available, directly determine the amount of settlement which will occur. No other factors operated with the consistency or effect of these.

Let us consider what happens when a fill is placed over a peat deposit. The top crust is first subjected to compression beneath the interior of the fill and to shear and tension at the edge. The communication of this compression to the interior of the peat is opposed by the resistance to flow of the peat or its internal friction. On account of the inertia of the peat particles, water is squeezed out of the material nearest the fill and the material is compacted, and this process extends with continually decreasing effect as distance from the fill increases until at some distance the peat is practically undisturbed.

Lateral flow of peat begins at an early stage in the placing of a fill and is due to the horizontal component of pressure due to the internal friction of the material and also to the inclined position of the settling load. Peat is forced outward and upward, until in the course of time a condition of equilibrium is reached.

It is easily understood, then, that the width of fill is a function of the amount of settlement which takes place. The fills in this investigation do not vary greatly in width, viz, from 24 to 30 feet, and so no account is taken of this factor. It is important, however, in considering the cause of the greater settlement in deeper deposits.

When deposits are relatively deep in proportion to their width, considerable vertical movement can take place before any marked reaction is experienced from the mineral subsoil below. In this case, lateral flow is comparatively small at first and a large amount of settlement is possible before there is much resistance; the elastic crust ruptures in tension and shear and the whole weight of the fill is suddenly thrown on the soft material below. This sudden application of weight very quickly compacts the peat below to a degree at which lateral flow begins in a considerable amount and we have a "sink hole." A portion of the peat flowing from beneath the load of earth is forced up at the sides. Due to the lighter weight of the peat as compared to the fill material, a correspondingly larger amount of peat must be raised to balance the heavier fill. Subsequent loading increases this heaving until the weight of the heaved peat brings about a condition of equilibrium.


YELLOW-GRAY SEDIMENT WITH CHARA AND SHELL-MARL ADMIXTURE
BROWN WOODY AND COARSE FIBROUS PEAT
THEORETICAL FILL SHAPE
BLACK WOODY PEAT WELL DECOMPOSED
BROWN FIBROUS PEAT
BLACK WOODY PEAT
BROWN SEDIMENTARY PEAT WITH FIBROUS ADMIXTURE
DARK GREEN SEDIMENT
BLUISH GRAY CLAY
SAND
FEET
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20
30
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70
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Fig. 1.—Fill of Normal Cross Section in a Peat Marsh About 22 Feet Deep from Surface to Clay

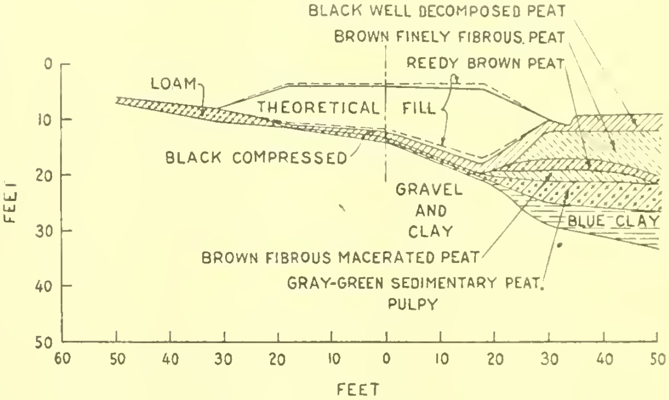


Fig. 2.—A Fill Partly on Upland Soil and Partly on Peat Marsh

SLOPE OF SOIL BENEATH PEAT BED AFFECTS FILL SHAPE

In the later stages of fill settlement the rate of settlement is very slow, but even a rate of a foot a year is decidedly objectionable. The rate progressively decreases until finally, after a long time, equilibrium is established. The length of time required to reach a stable condition in some cases is known to have been more than five years. We have arbitrarily established the rate of five-hundredths of a foot in 30 days as being the rate of settlement at which filling may be stopped. Since the rate will usually decrease progressively, this should give a settlement of less than 6 inches per year.

Unfortunately, at times, if the early settlement is not rapid, some later cause may accelerate the rate of settlement. Fills completed in the fall under earlier methods were quite apt to settle rapidly the following spring. The vibration of traffic also tends to increase the rate of settlement after a road is first opened. Large ditches cut along an old road are quite apt to start considerable settlement. As a matter of fact, unless settlement during filling has been accelerated in order to insure its completeness, some subsidence is almost sure to take place later.

A number of cross sections illustrating the various shapes taken by fills under different conditions are shown. Figure 1 shows a fill of normal cross section in a peat marsh about 22 feet deep from surface to clay. The theoretical shape of fill is shown by the dashed line and is obtained by a method described later. This deposit is of nearly equal depth on both sides of the fill and therefore the shape of fill is quite regular.

Figure 2 shows a fill which is partly on upland soil and partly over peat marsh. It will be noticed that the mineral subsoil slopes sharply toward the peat deposit and for this reason the proportion of the fill beneath the surface of the peat is considerably increased above that shown on the normal cross section. It will also be noticed that on account of the flow of the material only a very small amount of the peat remains under the fill, and that the layers to the right of the fill are considerably thickened by reason of their flow from beneath the fill and subsequent expansion due to the decrease of pressure.

Figure 3 shows a fill which is still settling into the peat marsh at a rapid rate. The slope of the bottom of the fill is quite noticeably parallel to the slope of the mineral subsoil beneath the light clay and most of the distortion of the peat layers has occurred on the low side of this mineral subsoil. It is quite evident that even with this considerable depth of peat beneath the fill, pressure is communicated throughout the entire mass.

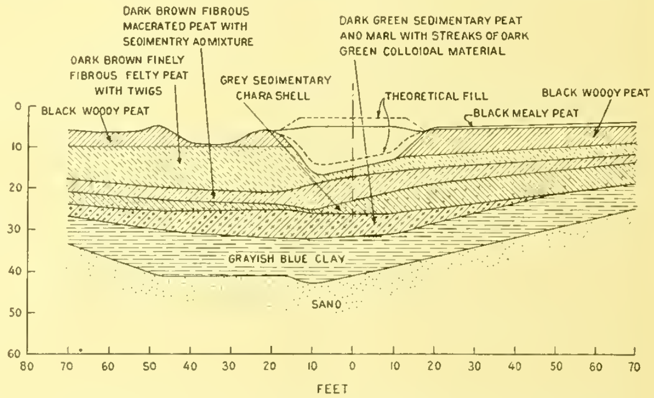
The effect of the slope of the mineral subsoil upon the direction of pressure is still more marked in Figure 4, where it will be noticed that, except immediately adjacent to the fill, practically all of the disturbance of the peat has taken place to the left of the center line, indicating that flow has largely occurred in this direction.

Figure 5 shows a fill which was placed adjacent to a railroad fill at the right, but not shown in the figure. In this instance it will be noted that the compacted peat to the right of the center line reduced the amount of fill necessary in a material manner. A very considerable amount of distortion of the layers is shown on the left of the figure. In this instance the peat is heaved above the normal level of the marsh some 10 or 12 feet and slopes away from the crest of the heaved material in a fairly uniform manner.

Figure 6 shows the deepest cross section encountered in this investigation, a fill with a depth of about 66 feet. The marsh, over which this fill was made, lies between two fairly large lakes, which are at present connected by a small stream. It is quite evident that the location of the highway in this particular instance was very unfortunate in that the center line of the highway at the cross section shown is practically on the center line of the old glacial drainage channel between the lakes in the early stage of their formation. A location of center line 30 feet to the right of its present location, which would have been possible, would have eliminated about 25 feet of fill. The entire amount of the lake clay which probably nearly filled the old channel, has been forced to one side by the load of fill above it. The irregular shape of the cross section on the right is probably due to filling a hole left by the peat heaving 8 or 10 feet at the side during construction.

METHOD OF PREDICTING FILL SETTLEMENT AND SHAPE DEVELOPED

From the whole series of cross sections of the character of those illustrated, most of which, however, are considerably more regular, diagrams giving the amount of settlement and shape of fill have been prepared. Depths of peat and marl on center line of flll were platted against the total depth to which the fill penetrated the peat and marl as measured from the original marsh surface. Three curves were then drawn, one representing the average of fills with a height above marsh level of 1 to 3 feet, one for 3 to 4 foot fills, and the third for fills from 4 to 5 feet in height. These three curves, shown in Figure 7, indicate that the amount of settlement increases uniformly but is not great up to a marsh depth of 20 feet. As the marsh depth increases from 20 to 26.5 feet the settlement increases rapidly and when the depth exceeds 26.5 feet the fill goes completely through the peat in every instance, so that the penetration is equal to the depth and the three curves become common and make a straight line with a slope of 45°. Therefore, for peat and marl depths of less than 26.5 feet the depth of lake clay need not be taken into account, except as a factor should settlement be different from the average, and anticipated settlement is read directly from the


DARK BROWN FIBROUS MACERATED PEAT WITH SEDIMENTRY ADMIXTURE
DARK GREEN SEDIMENTARY PEAT AND MARL WITH STREAKS OF DARK GREEN COLLOIDAL MATERIAL
DARK BROWN FINELY FIBROUS FELTY PEAT WITH TWIGS
GREY SEDIMENTARY CHARA SHELL
THEORETICAL FILL
BLACK WOODY PEAT
BLACK MEALY PEAT
BLACK WOODY PEAT
GRAYISH BLUE CLAY
SAND
FEET
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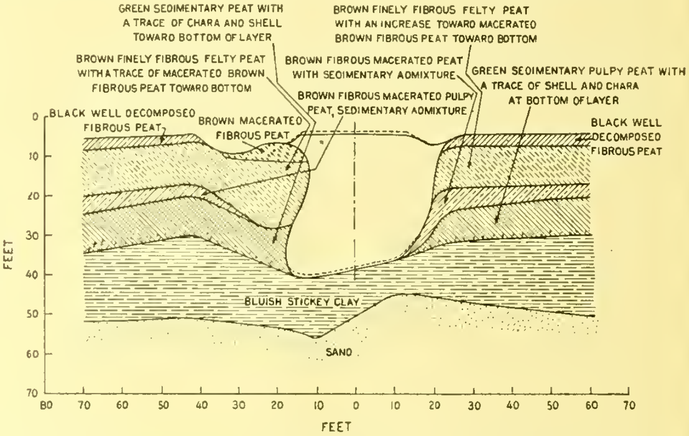
80 70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
FEET

Fig. 3.—Shape of Fill in Peat Considerably Affected by Slope of Mineral Soil at Bottom of Peat Bed


GREEN SEDIMENTARY PEAT WITH A TRACE OF CHARA AND SHELL TOWARD BOTTOM OF LAYER
BROWN FINELY FIBROUS FELTY PEAT WITH AN INCREASE TOWARD MACERATED BROWN FIBROUS PEAT TOWARD BOTTOM
BROWN FINELY FIBROUS FELTY PEAT WITH A TRACE OF MACERATED BROWN FIBROUS PEAT TOWARD BOTTOM
BROWN FIBROUS MACERATED PEAT WITH SEDIMENTARY ADMIXTURE
GREEN SEDIMENTARY PULPY PEAT WITH A TRACE OF SHELL AND CHARA AT BOTTOM OF LAYER
BROWN FIBROUS MACERATED PULPY PEAT, SEDIMENTARY ADMIXTURE
BLACK WELL DECOMPOSED FIBROUS PEAT
BROWN MACERATED FIBROUS PEAT
BLACK WELL DECOMPOSED FIBROUS PEAT
BLUISH STICKEY CLAY
SAND
FEET
0
10
20
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70
80 70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
FEET

Fig. 4.—Distortion of Peat Fill to the Left Due to Slope of Firm Soil Beneath Peat Bed

diagram. For depths of peat and marl over 26.5 feet the settlement will probably be the full depth of the deposit plus some penetration into the lake clay. The amount of penetration into the latter material is read directly from the curve marked lake clay, which has been plotted, using the combined depth of peat and clay as abscissas.

Suppose we have a peat marsh consisting of 30 feet of peat and marl and 5 feet of lake clay. The peat curve gives us complete penetration of the full 30 feet of peat and from the 35 feet total depth of the marsh deposit we read off from the clay curve 2 feet for clay penetration. There will then be a distance from bottom of fill to marsh surface of 32 feet. Again, if we had a peat deposit consisting of 38 feet of peat and 2 feet of lake clay we should get 38 feet of peat penetration plus 4 feet of clay penetration. The total, 42 feet, exceeds our total depth of deposit and the fill therefore stops at the marsh bottom with a 40-foot penetration. This last case is not common except in the very deep deposits, of 50 feet or more. It will usually be found that in marshes of such extreme depth with ordinary proportions of peat and lake clay, the fill goes to the very bottom.

No claim of scientific reason is made for the shape of the curves shown in Figure 7. They are purely empirical and represent merely the average behavior of a considerable number of fills. Extreme variations of actual from expected settlements as derived by this method have been found to be as much as 100 per cent for an indicated settlement of 5 feet, 60 per cent for 10 feet, 30 per cent for 20 feet, 20 per cent for 25 feet, 10 per cent for 30 feet, and less for greater amounts. Generally accuracy of the method is well within these limits.

The method of estimating fill quantities is illustrated by Figure 8. The typical shape shown was arrived at by drawing on cross-section paper all cross sections of equal depth, superimposed on each other, with the marsh level as a base line, and then fitting an average shape to them. A side slope of embankment above marsh level of 1 to 3 feet was used, since most of the fills studied had this slope. The fills quite generally slope inward from the outermost point of the embankment to a point directly beneath the shoulder line. The total area of fill is composed of two trapezoids and can be calculated by the following formula, the symbols representing distances as indicated in Figure 8.

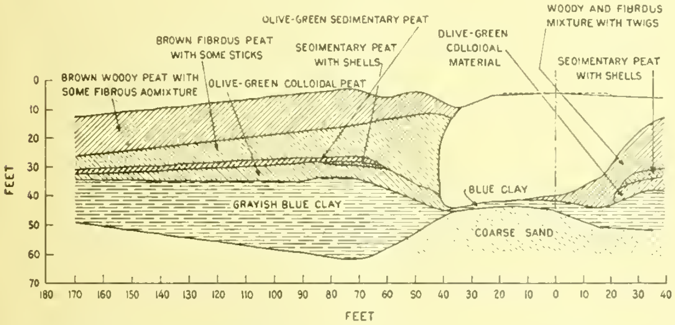

OLIVE-GREEN SEDIMENTARY PEAT
WOODY AND FIBROUS MIXTURE WITH TWIGS
BROWN FIBROUS PEAT WITH SOME STICKS
SEDIMENTARY PEAT WITH SHELLS
OLIVE-GREEN COLLOIDAL MATERIAL
SEDIMENTARY PEAT WITH SHELLS
BROWN WOODY PEAT WITH SOME FIBROUS ADMIXTURE
OLIVE-GREEN COLLOIDAL PEAT
BLUE CLAY
GRAYISH BLUE CLAY
COARSE SAND
FEET
0
10
20
30
40
50
60
70
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
10
20
30
40
FEET

Fig. 5.—A Fill Adjacent to a Railroad Fill at the Right But Not Shown. Compacted Peat on the Right Side Reduced the Amount of Fill Considerably

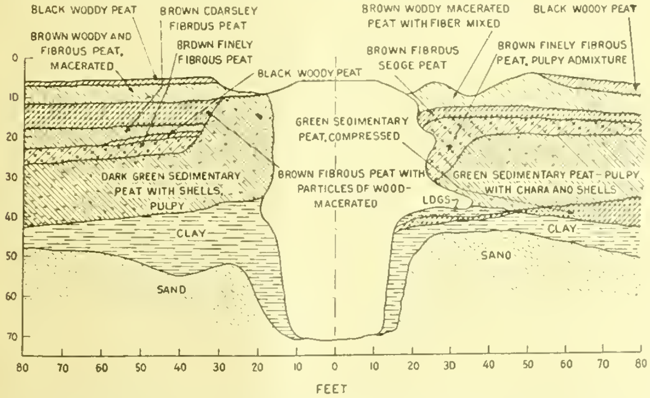

BLACK WOODY PEAT
BROWN COARSLEY FIBROUS PEAT
BROWN WOODY MACERATED PEAT WITH FIBER MIXED
BLACK WOODY PEAT
BROWN WOODY AND FIBROUS PEAT, MACERATED
BROWN FINELY FIBROUS PEAT
BROWN FIBROUS SEDGE PEAT
BROWN FINELY FIBROUS PEAT, PULPY ADMIXTURE
BLACK WOODY PEAT
GREEN SEDIMENTARY PEAT, COMPRESSED
DARK GREEN SEDIMENTARY PEAT WITH SHELLS, PULPY
BROWN FIBROUS PEAT WITH PARTICLES OF WOOD-MACERATED
GREEN SEDIMENTARY PEAT - PULPY WITH CHARA AND SHELLS
LOGS
CLAY
CLAY
SAND
SAND
FEET
0
10
20
30
40
50
60
70
80
70
60
50
40
30
20
10
0
10
20
30
40
50
60
70
80
FEET

Fig. 6.—An Unusually Deep Fill. Soundings Would Have Indicated This Condition and Permitted More Advantageous Location

$$\text{Area} = (W + 3F)\ (F + H)$$

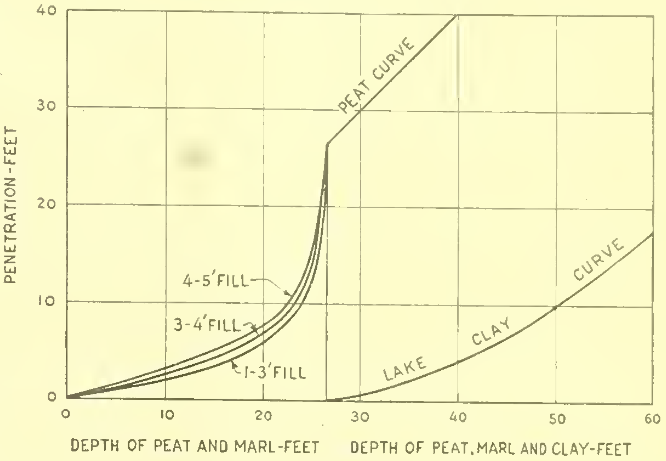


Fig. 7.—Diagram Giving Probable Depth of Penetration of Fills in Peat Marshes

In each case *H* must be determined by use of the curves shown in Figure 7. In Figure 8 the bottom of the predicted shape of fill is shown as a horizontal line and this was very nearly the case in most instances, the center of fill being only slightly lower than the outside. The depth of penetration, as indicated by Figure 7, takes account of these variations and gives the average depth to the bottom of the fill.

FILL SETTLEMENT SHOULD BE ACCELERATED

It may be surprising to note that the slope of the fill sides beneath the surface of the peat is toward the center of the road, but, with a little consideration, the reason for this is quite apparent. In the first place, the fill load as placed varies from zero at the toe of slope to its maximum at the shoulder line, and hence should produce a fill shape corresponding to this loading. Also, in a good many instances the fill has been carried across the marsh at less than full width while heavy settlement was taking place, and this produced a narrower fill at the bottom. Undoubtedly the drag of the compacted peat at the side also tends to narrow the fill width as it sinks. This is indicated by the fact that the center is quite generally lower than the rest of the cross section.

One of the striking features of this investigation was the irregularity shown in the topography of the marsh bottoms. In a number of instances a slight shift in line would have saved thousands of yards of earth in filling. It is therefore necessary when the location of the road requires the crossing of a peat marsh to map the topographical features of the marsh bottom quite accurately. This is more important than on upland soil because visual inspection of the proper location can not be made but must be determined by the topographical information secured from sounding.

In places where there is any considerable depth of peat it is highly necessary that settlement during filling be encouraged in every way possible. This settlement is bound to take place sooner or later, and it is important that it be practically complete before any rigid surfacing is placed. Construction of brush mats to float the fill is not advisable in the case of highway work where hard-surface pavement is to be placed. These mats only delay settlement and do not stop it. This is contrary to the practice of some railroads, but in railroad work it is a simple matter to add ballast and bring the track back to grade as settlement progresses without interrupting traffic. In case of any considerable settlement on a hard-surfaced highway, the subgrade must be refilled and a new surface placed. This always involves more or less interruption to traffic besides the greater expense of placing the new surface.

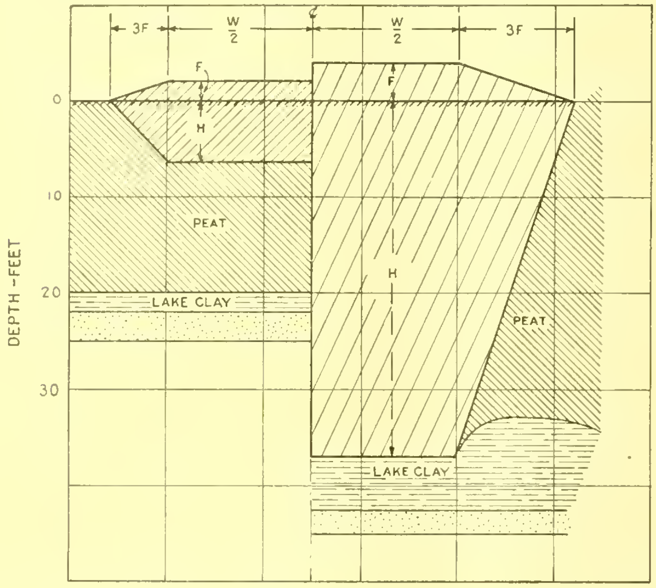


Fig. 8.—Diagram Illustrating Typical Shape of Peat Fills for Use in Estimating Earthwork Quantities

The length of the bog should be considered in choosing a method of filling. Settlement over short bogs, even in small amounts, is much more objectionable than on longer stretches because of the sharper deflection of the settled surface, and the liability to offsetting of the pavement slabs. A long gradual settlement of considerably greater amounts is not so noticeable either to the eye or in the riding qualities of the pavement. Of course, settlement over either long or short bogs is objectionable if the amount is great enough to interfere with the drainage. Settlement sufficient to cause trouble on this account is not likely to occur in marshes of less than 6 feet in depth.

STANDARD METHODS OF FILLING DEVELOPED

Figures 9 and 10 illustrate methods of filling over peat bogs now standard with the Michigan State Highway Department, one or another of which is used depending upon the length and depth of the bog to be crossed. Method No. 1 simply consists of placing the fill on the peat surface and is used where the marsh is less than 6 feet deep, more than 300 feet wide and undrained. The bog is long enough so that, if settlement should take place, it would be rather gradual, and the 24-inch height of fill above the original marsh will usually absorb this settlement without bringing the top of fill too close to the marsh surface.

Method No. 2 is used in drained peat marshes up to 6 feet in depth and over 300 feet in width. It is similar to method No. 1 except that the marsh drainage ditch goes entirely through the peat deposit. The ditch is excavated prior to the placing of the fill. This type of construction results in a much drier subgrade, and the lowered water table helps to prevent considerable settlement. This scheme should be used where conditions of the marsh drainage system require the use of ditches of considerable size.

Where bogs are less than 300 feet in width, and not more than 6 feet in depth, and even small settlement is undesirable, method No. 3 is used. This method requires that all the peat to a width 4 feet greater than the proposed pavement slab be excavated to marsh bottom, and the excavation filled with good earth. This method is not so expensive as it would seem, because the peat is very easy to move and can usually be disposed of with a very short haul. Peat excavation of this type is not practical for depths greater than 6 to 8 feet on account of the large amounts of peat which slide into the hole from the sides.

The above methods are not suitable for bogs much in excess of 6 feet in depth. In the deeper deposits settlement is almost sure to take place and should be aided while the fill is being placed. Method No. 4 in Figure 10 illustrates a Michigan practice in deposits over 6 and less than 20 feet in depth. In the first stages, two narrow fills, spaced 16 feet on centers, are carried across the marsh.

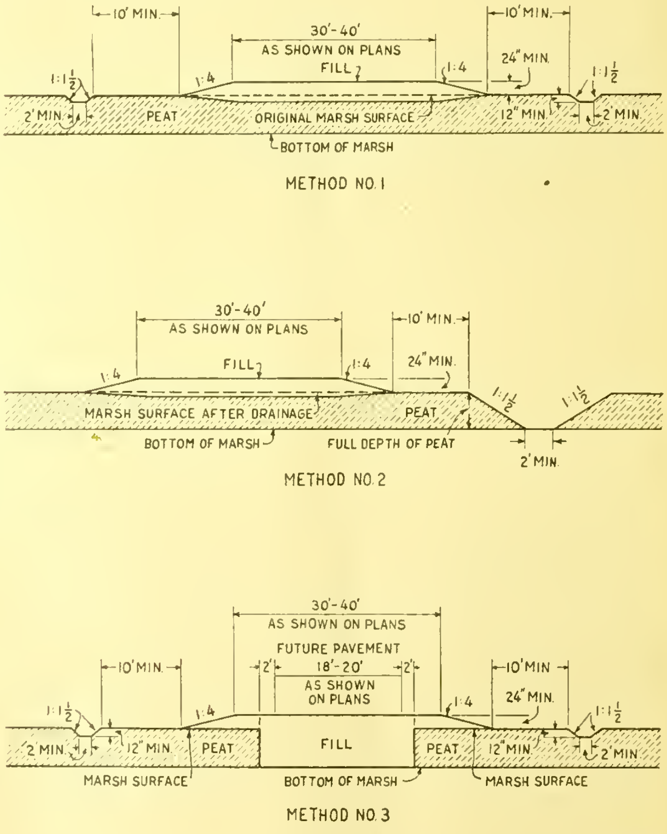


Fig. 9.—Standard Methods of Michigan State Highway Department for Filling Over Peat Bogs

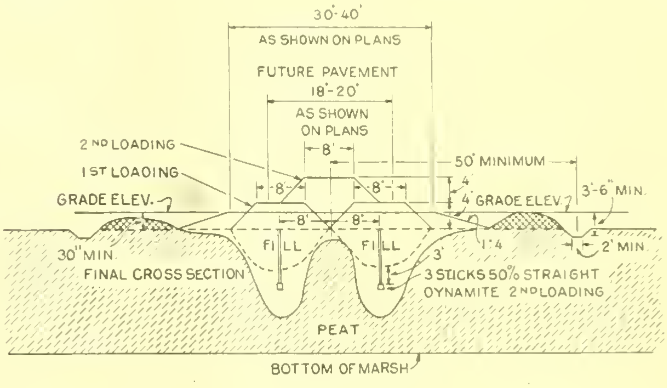
The supporting power of the peat crust is destroyed by blasting on the center line of each of the fills. The trenches thus made are backfilled with earth in 4-foot lifts and carried simultaneously across the marsh, trapping a certain amount of peat between the two fills. When the first two lines of embankment are complete and settlement has slowed up to a considerable extent, charges of dynamite are placed in the peat on 4-foot centers, from 2 to 3 feet below the bottom and on the center line of each embankment, and exploded. This second blasting drives the peat from beneath the wedges of earth and induces further rapid settlement. After refilling the settlement caused by the shooting, the second loading of earth is added in a 4-foot lift on the center line of the road. This second loading helps drive the wedges of earth still farther into the peat and insures a maximum compaction in the center so as to eliminate the possibility of center settlement resulting from the method used in the first loading. This loading is maintained by refilling the settlement until the rate of settlement is less than five-hundredths of 1 foot in 30 days. The excess earth is then spread to form the complete embankment.

Bogs from 6 to 20 feet in depth usually give more trouble than any others, because they rarely settle rapidly, unless excessively loaded, and surfaces are laid on fills long before complete settlement has been attained. It is much more difficult, in bogs of these depths, to secure complete settlement due to the early resistance of the peat to compaction; and it is for this reason that the two narrow wedges of earth are used, the effort being to secure as deep a penetration as possible. The final loading under this scheme gives a static load which is nearly twice the normal load, and it is felt that little danger of further serious settlement need be feared. To insure the acceleration of settlement, it is required to maintain the grade to within 1 foot of the designed elevation at all times.

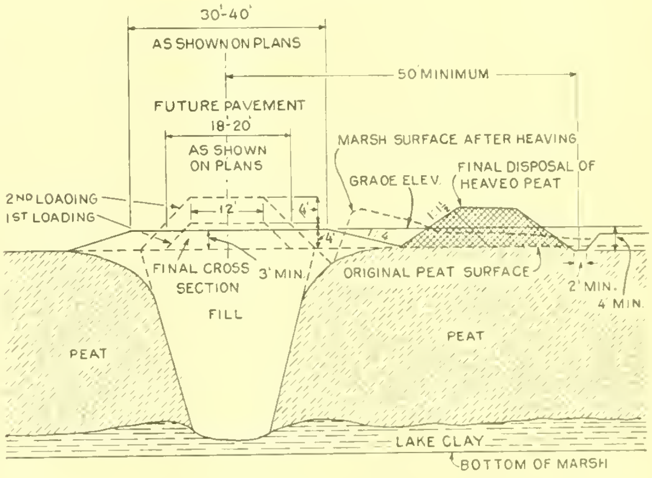
In long marshes, in order to avoid long backing by trucks in end dumping, provision is made for shooting part of the fill at a time. It is believed, however, that at least a hundred feet of fill should be held for settling at all times before any shooting is done.

For depths of peat over 20 feet, the center loading is used at once after the surface of the peat has been broken up by dynamite. A fill 12 feet wide at the top and 4 feet high is carried across on the center line. This fill is maintained at grade as nearly as possible until it is entirely across the space over which this method is to be used. As soon as it can be maintained at grade entirely across the marsh, a second 4-foot lift is made without any preliminary shooting. It is felt that with a fill width of this amount, dynamite would not displace the peat sufficiently to justify its use. In addition to this, the greater depth of the marsh makes settlement so much easier that the loading alone produces sufficient settlement. After the settlement has slowed down to a rate of five-hundredths of 1 foot in 30 days, the fill is unloaded and the excess material spread to form the remainder of the cross section. The compressed peat on both sides of the narrow fill will usually support the additional width with little settlement.

Any combinations of these methods may be used in crossing peat marshes of considerable length, suiting the method to the depth and width. The first requirement in all marsh filling where pavement is to be placed is to insure settlement, and not attempt to avoid it. Unless the fill has become stabilized, this settlement may occur over long periods and the former practice of attempting to "sneak across" with light loadings has been responsible for a great deal of maintenance expense in locations of this character. The proper time to take care of this settlement is during construction, as it is extremely difficult to maintain in a satisfactory way a surface which is continually settling.



METHOD NO.4



METHOD NO.5

Fig. 10.—Standard Methods of Michigan State Highway Department for Filling Over Peat Bogs

FILL CONSTRUCTION SHOULD PRECEDE BRIDGE CONSTRUCTION

If any bridges are to be placed over streams through deep peat deposits, the approaches should always be completed first. Filling should continue toward the center of the stream so that excavation for footings will occur in the filled earth. In some cases, where streams are small, it will be found advisable to carry this filling completely across the old stream bed. The expense of later cleaning out the earth above stream bottom and taking care of the stream flow during filling is money well spent.

The above may seem ridiculously obvious to many engineers, but the frequency with which bridge failures have occurred during the past because of ignorance of the troubles which may arise through neglect of this item justifies it. When a bridge rests on piling with lengths of over 10 feet penetrating soft peat before

(Continued on page 247)

DETERMINATION OF CONSISTENCY OF SOILS BY MEANS OF PENETRATION TESTS

Reported by Dr. CHARLES TERZAGHI, Massachusetts Institute of Technology, Research Consultant to the Bureau of Public Roads

When walking over the ground, one's foot may either sink into the ground to a depth of more than an inch, the soil being squeezed out laterally under pressure or the foot may leave hardly any impression with many intermediate possibilities. Judging from the effect of the pressure exerted by the foot, we say, that one soil is less consistent than the other.

The effect of the pressure exerted by the foot is obviously connected with what is known as the bearing capacity of the soil. Since, in addition, it is connected with what is commonly called the consistency, both engineers and geologists have attempted to establish some scale for expressing the consistency of soils for practical purposes. The basic idea is this: First to devise some simple standard test, whose outcome depends on the consistency of the material and then to correlate the *consistency coefficient* obtained by means of such a standard consistency test with the bearing capacity of the soil. This plan seemed to be particularly promising in all those cases, where the penetration of the loaded area is due merely to a lateral flow of the soil—e. g., penetration of a loaded area into soft fat clay.

The possibility of expressing the bearing properties of a soil by a *consistency coefficient* would certainly represent an ideal solution of an important problem. The purpose of this paper is to discuss related attempts and their chances for success.

TERMS DEFINED

The term *consistency* is by no means physically as clearly defined as are the terms *viscosity* or *hardness*. As it is commonly used, the term *consistency* means the resistance of a material to flow, yet resistance to flow is in itself a rather complex property. Suppose we have a mass of paraffin and a mass of soft clay, both with a plane, horizontal upper surface. Then we select a weight, which, if placed on top of the clay, is heavy enough to produce a marked impression without completely sinking into the material. If we take another weight, exactly alike and put it on the surface of the paraffin, we will notice that the impression produced by the weight is far less significant. Yet, if we observe the two weights during a sufficiently long period, we will notice that the speed with which the weight penetrates the clay rapidly decreases and after a certain time its downward movement stops completely. If the weight is carefully removed and again applied, the increase in penetration will be exceedingly small. On the other hand the speed with which the weight penetrates the paraffin remains fairly constant, although it is very small, even at the beginning. As a consequence, after a certain length of time the penetration of the weight into the paraffin may considerably exceed the penetration produced in the clay, although the initial penetration was very much smaller. If the weight is removed and again applied, the penetration will continue to increase about at the same rate as it did at the beginning of the process. The question arises: Which one of these two materials is the more consistent?

After a short period of time, the depth of penetration in paraffin will be very much smaller than in clay. After a sufficiently long period the opposite will be true. In order to understand this apparent contradiction, we must consider the following facts: The speed with which the weight penetrates the material depends on viscosity, a property common to both solids and liquids. If a material has only viscosity, it has no static bearing capacity, though in every other respect it may exhibit all the properties of a solid body—e. g., sealing wax or paraffin. It flows under any pressure, although it may flow very slowly. Certain other materials, as, for example, soft clay, have, compared with sealing wax, a rather low viscosity, which means that the soft clay is very much more *fluid* than is the paraffin. Yet in contrast to the sealing wax, under very low pressures the clay does not flow at all. In order to start the flow, the pressure must exceed a certain limiting value, known as the *yield point*. The yield point represents the ultimate bearing capacity of the clay. Although the soft clay is much more fluid and much less viscous than the paraffin, yet its yield point is very much higher. For this and similar reasons, the consistency of a material can not possibly be expressed by a single figure. It must be represented by a graph.

However, when dealing with subgrades we are merely interested in what the subgrade can stand without the deformation going on indefinitely. Hence, if we use the term *consistency* in connection with foundation problems at all we should confine it strictly to the maximum pressure under which the material is still apt to to come to equilibrium, that means to the yield point. This statement determined the fundamental requirement for standard consistency tests for subgrade or foundation purposes. The figures furnished by the tests should be in a constant and well-defined relation to the yield point of the material. At the same time it should be kept in mind that such figures inform us about one phase only of what the physicists call the consistency. The other phase of this property, viz, the behavior of the material in the state of continuous flow, under a pressure above the yield point, may be interesting from a scientific point of view, but it has no bearing on our foundation problems.

It is obvious, that the yield point, that means the maximum pressure per unit of area under which the material is still apt to come to a state of equilibrium, depends on its state of confinement and on the way in which the pressure acts. Thus, for instance, the yield point of a thin layer of soft clay inclosed between a hard road surface and a more consistent under-ground is very much higher than the yield point of the same material if only loaded over an area of several square inches or if tested in an unconfined state. Yet these conditions are a matter of detail and will be considered later. The essential point merely resides in the possibility of an ultimate state of equilibrium under a locally applied pressure which, in the case of sealing wax or paraffin, does not exist.

The most logical procedure for rapidly evaluating the yield point of a material seems to be that of loading the surface of the material with a cone of standard weight. The deeper the cone penetrates into the material the smaller becomes the pressure per unit of surface of contact between the cone and the material. Hence the depth at which the cone refuses to penetrate farther into the material can be used as a scale for expressing the resistance of the material to penetration. Due to the apparent efficiency of the procedure, the idea of using it was independently conceived by investigators working in different parts of the world; by the Swedish Railroad Commission for the investigation of earth-slides, operating during the years 1914–1922; by the writer in Constantinople in 1923; by Dr. E. M. Kindle in Ottawa, Canada (1925), and quite recently by the committee for the standardization of loading tests of the Austrian Society of Civil Engineers.

The following paragraphs contain a brief description of some of the experimental facts brought out by these tests.

TESTS OF THE SWEDISH RAILROAD COMMISSION[1]

For testing the consistency of drill samples the Swedish Railroad Commission used cones with weights of 60 and 100 grams and with vertex angles of 60°. In addition to this, systematic investigations were carried out with cones having weights of 10, 30, 200, and 300 grams, and with points of 30° and 90°. The material was contained in a soil cup, with a diameter of 6 centimeters. Prior to the test a cone was fixed in such a position that the point just touched the surface of the material. The cone was released and allowed to freely penetrate the material and the measurement of the depth of penetration was made as soon as the cone came to rest.

The depth of penetration of the 60-gram (or of the 100-gram) cone served as a basis for computing the consistency factor of the material. The consistency factor is the weight of the cone required for producing a standard penetration of 4 millimeters. Determination of the consistency factor was made by means of a diagram obtained in the following manner:

Suppose we increase the water content of a sample of clay by adding water to it. As a result of this procedure both the softness of the clay and the depth of penetration of a cone with a standard weight of 60 grams increases. Then let us plot a diagram, wherein the abscissas represent the softness of the clay and the ordinates the depth of penetration of cones. If we consider the depth of penetration of the standard cone with a weight of 60 grams as a scale for the softness of the clay, the penetration curve of the standard cone will be a straight line, passing through the origin and bisecting the angle between the two axes. In order to find the penetration curves for cones 1, 2, 3, etc., with a weight other than 60 grams, we take a sample with a defined water content, into which the standard cone penetrates to a depth of say t millimeters, determine the depth of penetration t_1, t_2, t_3, etc., of the other cones, and plot all the values thus obtained on the same ordinate which corresponds to the depth of penetration t of the standard cone. By repeating the same procedure on samples into which the standard cone penetrates to a depth of t', t'', etc., we obtain a series of points, t_1, t_1', t_1'', etc., t_2, t_2', t_2'', etc. These points determine the penetration curves for the cones 1, 2, 3, etc. From this graph one can easily derive another one, in which the abscissas represent the depth of penetration of the standard cone, whereas the ordinates represent the weight of cone required to produce a standard penetration of 4 millimeters. Since it was agreed to call these weights *consistency factors*, the curves thus obtained are called the *consistency curves*. By means of the consistency curves, the consistency factor of any sample can be determined, provided the depth of penetration of the standard cone is known. Hence the reliability of the procedure depends essentially on the reliability of the consistency curves.

The results of the investigations, which obviously involved a very considerable amount of time and labor, disclosed some facts which seriously affect the value of the method. It was found that the depth of penetration of the standard cone into the undisturbed material was invariably smaller than the penetration of the cone into the same sample after the sample was remoulded, the water content and the density of the sample remaining unchanged. This means that the consistency of the material depends not only on the water content, but also on the history of the sample. Furthermore, the ratio between the consistency factor of the undisturbed and the disturbed sample was found to be very different for different materials. Hence no conception of the consistency of a soil deposit can be derived except from tests performed on undisturbed samples. Finally it was found, that any consistency curve is valid for one definite family of soils only. Different soils have different consistency curves.

An analysis of the last fact mentioned above led the writer to the following conclusions: If the depth of penetration of the standard cone depended on nothing but the *hardness* of the material, the consistency curves should be valid for all fine-grained soils, irrespective of their physical character. Hence the lack of universal validity of these curves indicates that the depth of penetration depends not only on the hardness but also on a foreign factor which has nothing to do with consistency. This other factor seems to reside in the friction between the soil and the sides of the cone, combined with purely viscous resistances acting while the material is flowing.

YIELD POINT RELATED TO CUBE STRENGTH

Before passing to the investigations of the writer, it seems advisable to insert a brief statement concerning the theory of plastic flow, this theory being essential to a clear understanding of the nature of the processes involved in the cone tests. The theory of plastic flow produced by local application of an external pressure has been worked out during the last few years by Kármán, Prandtl, and Hencky.[2] From the investigations of these authors, we know that the hardness of plastic materials (resistance against penetration of a solid body into the material) is a simple multiple of the cube strength of the material. In turn, by definition, the hardness of plastic materials corresponds to the yield point, because under a pressure greater than the hardness the material flows. Hence the yield point and the cube strength of plastic materials are connected with each other by a simple and well-defined relation. This theoretical conclusion has been repeatedly verified by experiment. Thus, according to theory, the resistance of a material against penetration of a body with a long, narrow cross section should be equal to 2.57 times the cube strength of the material, regardless of whether the material is hard, like a metal, or as soft as putty. Tests performed with steel edges pressing against the plane surface of wrought iron showed that the ratio between resistance against penetration and cube strength was, for this particular case, equal to 2.31, against the theoretical 2.57.

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[1] Statens Järnvägars Geotekniska Kommission 1914-1922, Stockholm, 1922.

[2] L. Prandtl, Über die Härte plastischer Körper. Nachrichten von der K. Gesellschaft der Wissenschaften zu Göttingen, Math. phys. Klasse, 1920, and L. Prandtl, Anwendungsbeispiele zu einem Hencky' schen Satz über das plastische Gleichgewicht. Ztschr. für angewandte Mathematik und Mechanik. Bd. 3, Heft 6, 1923.

Recognizing the importance of the relation between cube strength and consistency (hardness, resistance against penetration, yield point, etc.) the writer started in 1920 to develop a method for measuring the cube strength and the elasticity of very soft materials. The results furnished by the tests were intended to be used in estimating the bearing capacity of natural soil deposits. The test method proved to be successful. However, the time required for testing one sample amounted to at least two hours. In order to simplify the procedure, the writer started in 1923 to investigate whether or not it was possible to compute the compressive strength of any soft material from the penetration of a cone with a standard weight, dropping from a standard height, or from the penetration of a cone forced into the material by a static pressure.

The investigations were carried out with a set of samples, extracted with drills from a mud deposit located at the Golden Horn in Constantinople, on similar sets of samples from drill holes through glacial clays in Detroit, Mich.[3] and on samples obtained from a soft deposit of glacial clay in Cambridge, Mass.[4] The samples were obtained by means of special samplers attached to the lower end of a set of rods. Whenever the ground encountered in the test boring seemed to change, the sampler was forced into the ground to a depth of 2 to 3 feet below the bottom of the drill hole. Then the sampler was withdrawn and detached from the rods. Finally both ends of the sampler were covered with paraffined cardboard and sealed by dipping each end three times into liquid paraffin, so as to keep the water content of the sample unchanged.

Two types of apparatus were used for making the tests: a Vicat needle device in which the needle was replaced by a cone with a 90° point, having a standard weight of 112 grams, dropping freely from a standard height of 4 centimeters (apparatus *A*); and a lever device with an automatically sliding weight, acting on a cone with a 90° point, whose point, at the outset of each test, just touched the plane surface of the sample (apparatus *B*).

The investigations included the following types of tests: Penetration tests with both apparatus *A* and *B* on undisturbed samples (soil still contained in the sampler), on disturbed samples (soil taken out of the sampler and transferred to a cylindrical mould), and finally cube tests, performed for the purpose of finding out whether or not a definite relation exists between the cube strength of the materials and the figures furnished by the penetration method.

Figure 1 shows the results of tests obtained by means of apparatus *A* on remoulded samples from the mud deposits located at the shore of the Golden Horn in Constantinople. Since the material was very soft, it was transferred from the samplers into a cup with a depth of 1½ inches and a diameter of 3 inches. The ordinates represent the cube strength of the samples, determined by unconfined compression tests. When selecting the abscissas for the diagram, the writer was guided by the following consideration:

Let

t be the penetration of the dropping cone in millimeters

r the radius of the hole, punched by the cone

q the cube strength of the material, and

Q the force equivalent to the impact produced by the dropping cone.

Since the vertex angle of the cone was equal to 90°, we have

$$r=t$$

and the vertical projection of the area over which the force Q acted was equal to

$$\pi r^2=\pi t^2$$

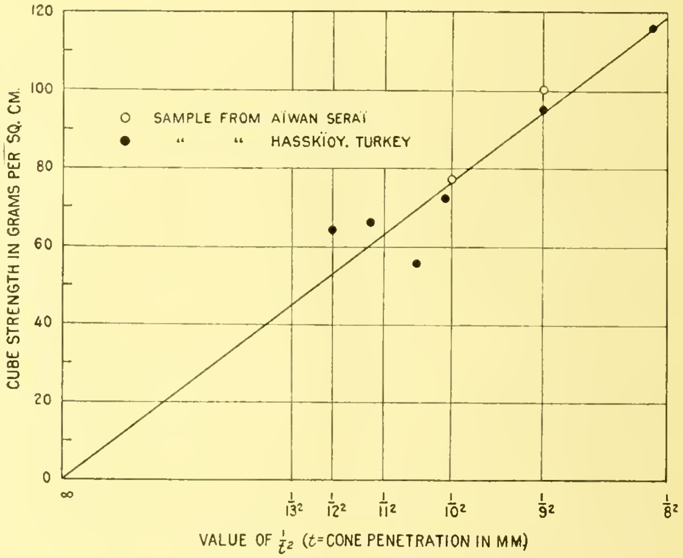

FIG. 1.—RELATION BETWEEN THE CUBE STRENGTH OF MUD SAMPLES AND THE CORRESPONDING PENETRATION t

According to the theory of plastic flow, the yield point is equal to

$$c_1q$$

wherein c_1 is a constant for all plastic materials. The cone ceases to penetrate the material as soon as the pressure per unit of area becomes equal to the yield point. Hence, combining the aforementioned relations, we obtain

$$\frac{Q}{\pi r^2}=\frac{Q}{\pi t^2}=c_1q$$

or

$$q=\left(\frac{Q}{\pi c_1}\right)\frac{1}{t^2}=\text{constant}\times\frac{1}{t^2}.$$

If the formula is correct the relation between the cube strength and the quantity $\frac{1}{t^2}$ should, in a diagram, be represented by a straight line. For this reason we have plotted in the diagram the values $\frac{1}{t^2}$ as abscissas. As a matter of fact, the points obtained by actual observation are very nearly located on a straight line, which indicates that the cone test furnished fairly reliable values for the relative "consistency" of these particular mud samples.

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[3] Tests made by Metcalf & Eddy, consulting engineers, Boston, Mass., according to instructions furnished by the writer.

[4] Tests by E. S. Sheiry, Massachusetts Institute of Technology, under the supervision of the writer.

Figure 2 shows the results of similar tests performed on drill samples from the blue clay deposit located beneath the Massachusetts Institute of Technology in Cambridge, Mass. The apparatus used for these tests was identical with the one used for the tests of Figure 1. The procedure was as follows: After removing the seals from the two ends of the sampler, a piston was forced into the sampler from one end, thus forcing about one-half inch of the material out of the tube. After removing the material projecting out of the tube, the surface thus exposed was smoothed off by means of a straightedge, and the cone was dropped in its center. This test furnished the depth of penetration in the "undisturbed" sample. Then an additional quantity of about 1 cubic inch of clay was forced out of the sampler, and molded into a cylindrical mold with a height of 1 inch. The cone test performed on the smooth top surface of this sample furnished the depth of penetration in the "disturbed" sample.

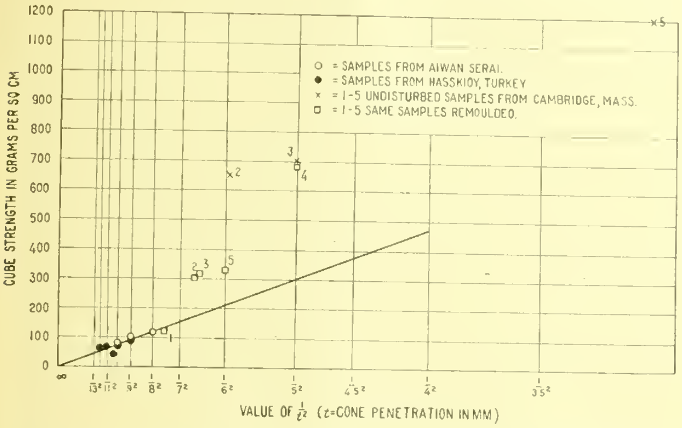

FIG. 2.—RELATION BETWEEN THE CUBE STRENGTH OF CLAY AND MUD SAMPLES AND THE CORRESPONDING PENETRATION *t* OF A STANDARD CONE, FREELY DROPPING FROM A STANDARD HEIGHT OF 4 CENTIMETERS

RESULTS OF CONE TEST FOUND TO BE MISLEADING

If the penetration of the cone depended on nothing but the yield point of the material, the points obtained from the tests should be located on the straight line determined in Figure 1 and shown also in Figure 2. Figure 2 shows, however, that this is by no means true. The points are scattered over the area in a rather erratic manner. In contrast to the Constantinople samples, the nature of the material of the Cambridge samples varied between rather wide limits. The observed facts suggested the following conclusions: The relation between the yield point and the penetration of a freely dropping cone is very different according to the nature of the material. The results furnished by the test may be misleading, unless the test is used for estimating the relative consistency of materials with a very similar character.

On account of the rather discouraging results with apparatus *A* the construction of apparatus *B* was decided upon. The material was prepared in the same manner as for the former tests. The force acting on the cone increased from zero at a constant rate of 30 grams per second. Every fifth second the depth of penetration of the cone was observed and recorded. Figure 3 shows the result of tests performed on an undisturbed and on a disturbed sample of a pale blue, glacial clay from a depth of 90 feet below the surface, with a liquid limit of 48.4 per cent, a plastic limit of 24 per cent, and a water content of 44 per cent. Figure 4 shows the results of similar tests performed on a clay from a depth of 44 feet, liquid limit of 49.7 per cent, plastic limit of 23 per cent, and a water content of 40 per cent.

The relation between the load acting on the cone and the corresponding depth of penetration should be as follows:

Let

Q be the force acting on the cone,

t the depth of penetration of the cone in millimeters, and

$r = t$, the radius of the vertical projection of the area over which the force Q acts.

According to the theory of plastic flow, the yield point should be independent of the area over which the pressure acts. Hence, if we plot the values of Q as abscissas and the values of t^2 as ordinates, the points obtained from each test should be located on a straight line, because

$$\frac{Q}{\pi r^2} = \frac{Q}{\pi t^2} = \text{constant}$$

or

$$Q = \underset{\pi\times}{(\text{constant})} \times t^2 = c_1 t^2$$

Figures 3 and 4 actually show that the straight line relation expressed by this formula approximately exists. In addition to this, they show in a very striking manner the importance of the influence on the yield point of remolding the samples of unaltered

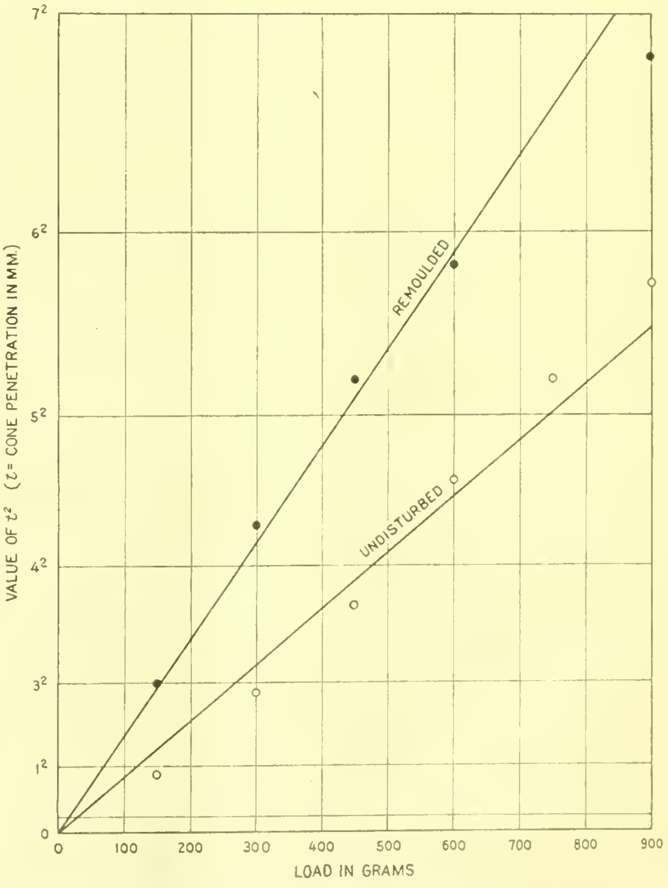

FIG. 3.—RELATION BETWEEN WEIGHT ACTING ON CONE AND SQUARE OF PENETRATION OF THE CONE INTO SAMPLE NO. 2

water content. The abscissa of the intersection between the penetration lines and a horizontal line with ordinates equal to 4^2 obviously corresponds to the consistency factor of the Swedish Railroad Commission.

YIELD POINT OF UNCONFINED SPECIMEN

In order to correlate the results of cone penetration tests with the yield point of the material, compression tests have been made on unconfined, cylindrical specimen of different clays. Figure 5 shows the results of the compression tests performed with undisturbed and the disturbed materials of Figure 3, and Figure 6 the results of similar tests made on the materials of Figure 4. The tests were made on cylinders 1 inch high, with a diameter of 1 inch. During the tests the cylinders were surrounded by a humidifier, and the remolding was done so rapidly that the water loss due to evaporation associated with the remolding process was practically negligible.

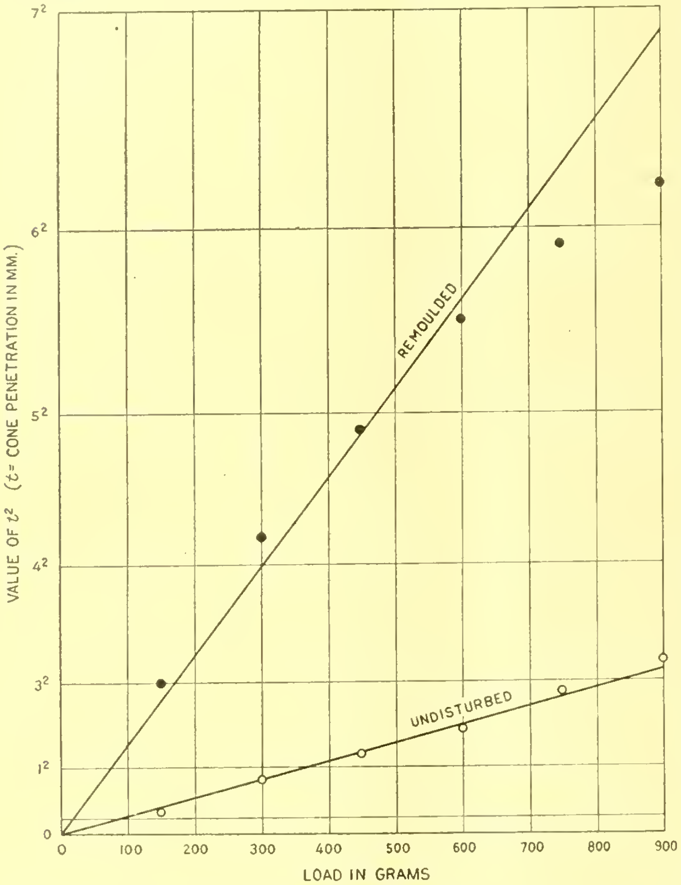

FIG. 4.—RELATION BETWEEN THE WEIGHT ACTING ON CONE AND SQUARE OF PENETRATION OF THE CONE INTO SAMPLE No. 5

The load was changed (increased or decreased) by equal increments in equal time intervals at such a rate, that every test lasted about two hours, the time required for the time observations *a*, *b*, *c*, etc., at constant load excluded.

If in such a test a new increment is added to the load, there follows an almost instantaneous compression. Then, as time goes on, the compression gradually increases at a decreasing rate, until finally equilibrium is reached, the time increase of compression becoming equal to zero. On the other hand, on removing precisely the same load increment, the specimen almost instantaneously expands. As time goes on, the expansion continues at a decreasing rate, until finally, after a couple of hours, expansion ceases. These phenomena are characteristic of the effect of pressure below the yield point and at the same time correspond in every respect to the effect of pressures on solid bodies. Thus, from the results of experiments with various metals, Michelson[5] concluded that a change in stress of a solid body involves four different strain effects: (1) Rapid elastic yield; (2) rapid inelastic yield (lost motion); (3) slow elastico-viscous displacement; and (4) slow viscous (nonreversible) displacement. The relation between time and each one of these four types of strain effects in solid bodies closely resembles those which exist for clays and similar nonliquid materials, regardless of how stiff or fluid they may be, provided the pressure which causes the strain effects be below the yield point. A discussion of the causes of the lag (retardation) of the strain has been presented elsewhere.[6] For our purposes it is sufficient to retain the following two facts:

The application of a pressure on clay always involves a viscous flow, regardless of whether the pressure be below or above the yield point.

If the pressure is below the yield point, the speed of flow decreases at constant pressure until it finally becomes equal to zero.

The maximum pressure (or, in the case of cube tests, the maximum load per unit of area) under which a state of final equilibrium can still be reached, corresponds to the *yield point.* Hence the time-compression curves for loads below the yield point approach a horizontal tangent, while the corresponding curves for loads above the yield point have no such tangent.

In Figures 5 and 6 the yield point corresponds to the abscissa of the vertical tangent to the pressure-compression curve marked "T" because the pressure compression curve joining this tangent means infinite increase in compression at no increase in load.

Tests can not be carried to the point where the compression curve becomes vertical but the position of the vertical asymptote can be estimated with a sufficient degree of accuracy. In the right lower corner of Figures 5 and 6, the increase of compression at constant load for different loads are shown for both undisturbed and remolded samples.

The degree of elasticity of the samples is determined by the reversible part of the compression and can be expressed by the modulus of elasticity, which in turn is equal to the cotangent of the angle formed between the pressure axis and the hysteresis loops.

The samples represented by Figures 5 and 6 had the following characteristics:

	Modulus of elasticity	Yield point	Ratio of yield point to modulus of elasticity
	Kg. per sq. cm.	*Kg. per sq. cm.*	
Sample No. 2 undisturbed	37	0.65	0.0176
Sample No. 2 remolded	25	.30	.0120
Sample No. 5 undisturbed	66	1.20	.0182
Sample No. 5 remolded	25	.33	.0132

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[5] The Laws of Elastico-Viscous Flow, by A. A. Michelson, Proceedings of the National Academy of Sciences, May 15, 1917.

[6] Terzaghi, Erdbaumechanik, Wien, 1925.

COMPRESSIVE STRENGTH AND CONE PENETRATION

The results of the cube tests furnished the basis for correlating the yield point with the data furnished by the penetration tests. From these results, the relation between the cube strength q of the material and the unit resistance $p=\frac{Q}{\pi r^2}=\frac{Q}{\pi t^2}$ to the penetration of the cone (yield point) was found to be as follows:

Sample	q	$p=\frac{Q}{\pi t^2}$	$\frac{p}{q}$
	Gms. per sq. cm.	*Gms. per sq. cm.*	
No. 2, undisturbed	650	956	1.47
No. 2, remolded	300	548	1.83
No. 5, undisturbed	1,200	2,960	2.47
No. 5, remolded	330	600	1.82
No. 3, undisturbed	700	1,310	1.87
No. 3, remolded	320	711	2.23

If the test with apparatus *B* were an ideal consistency test, the ratio $\frac{p}{q}$ should be a constant and entirely independent of the nature of the material. In spite of all the tests having been made with blue glacial clay, the ratio varies by about ±25 per cent, the average value being equal to 1.95.

The most conspicuous result brought out by the tests concerns the wide range in variation of the ratio between the cube strength of disturbed and undisturbed samples. The cube strength of the remolded samples ranges anywhere between 0.55 and 0.25 of the cube strength (yield point or consistency) of the undisturbed samples. Hence there seems to be no chance to evaluate the actual consistency of a soft, water-bearing deposit, except by experimenting with undisturbed samples, and the technique of securing undisturbed samples represents as important a problem as the technique of testing the samples.

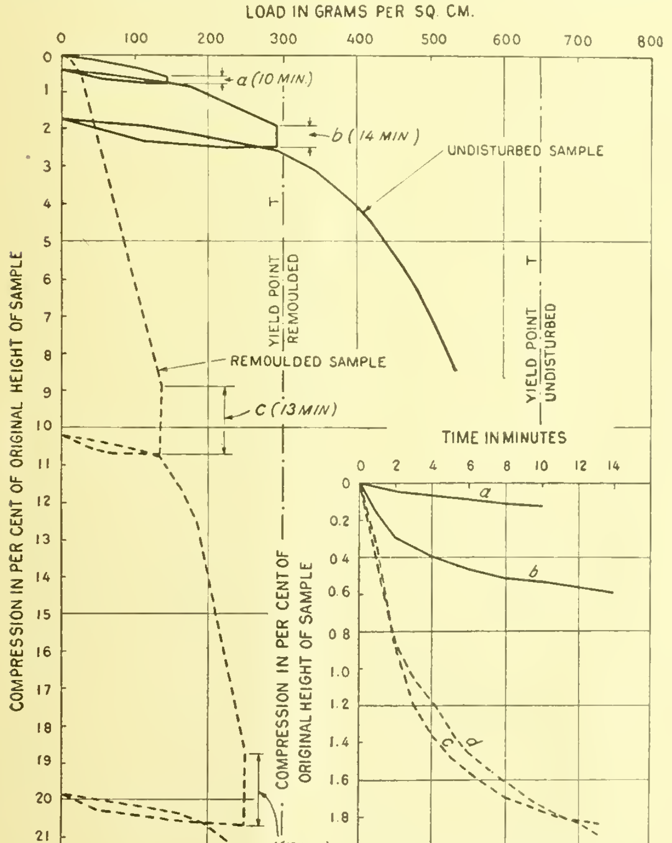

Fig. 5.—Compression Test on Unconfined Specimen of two Identical Clay Samples (No. 2) With Identical Water Content one of Which was in the Original State and the Other Remolded

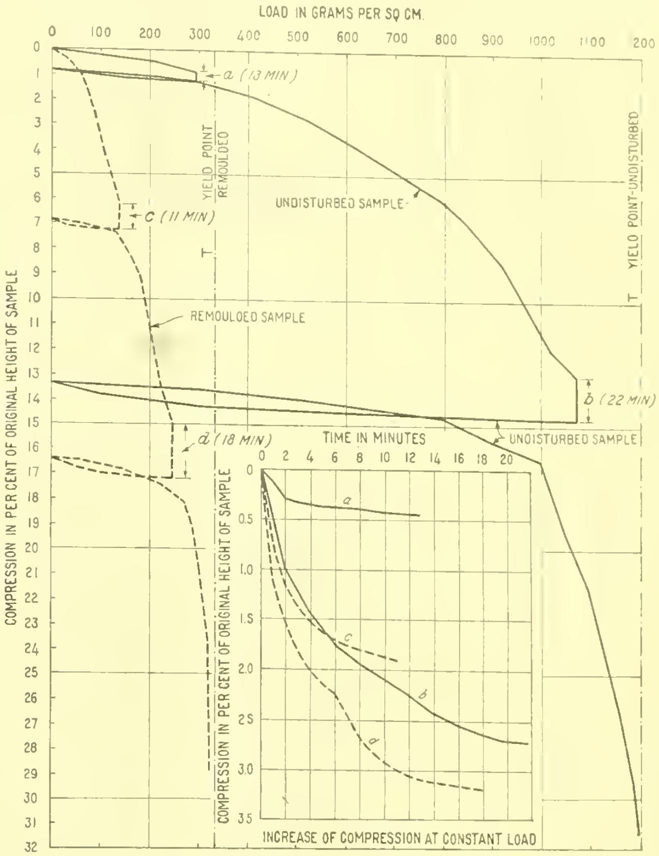

Fig. 6.—Compression Tests on Unconfined Sample of Two Identical Clay Samples (No. 5) With Identical Water Content One of Which was Undisturbed and the Other Remolded

FRICTION AFFECTS VALUE OF KINDLE'S TEST

In contrast to the writer, Doctor Kindle attacked his problem from a purely empirical point of view. He compared materials ranging in hardness between talcum and cohesionless sand with the successive stages through which a mixture of neat cement and water passes during the process of setting, and tried to work out an experimental method for correlating the consistency of any soft material with the corresponding consistency of a mixture of neat cement and water. This method of attack is very original and appealing on account of its apparent simplicity. It furthermore differs essentially from the methods used by the Swedish Railroad Commission and by the writer, inasmuch as it is intended to include not only clay and mud, but all other kinds of soft materials, plastic and brittle. It involves, however, from the very beginning a serious difficulty. Consider, for example, the two following materials: A sand whose grains are slightly cemented together and a soft clay. Both materials may have, according to any consistency test (cube test, bearing-capacity test, cone-penetration test, etc.) precisely the same resistance. Nevertheless it would be most confusing to consider the consistencies of these materials as equivalent, because there is obviously a second factor which must be taken into consideration, the deformability of the material According to the degree of deformability, the mechanical behavior may or may not be governed by the laws of plastic flow, a difference which is at least as important as a difference in consistency. Hence the consistency of the materials can not possibly be represented by a single figure, even in the restricted sense of the word, as used in this paper.

Disregarding this initial difficulty, we may pass to the test method itself.[7] In order to compare the consistency of two materials, Doctor Kindle measures the penetration of a one-half inch rod with a sharp point, freely dropping from an elevation of 10 inches. Upon analyzing the process of penetration, one realizes that the depth of penetration depends not only on the cohesion of the material, but also on the friction between the surface of the rod and the adjoining material. This second factor is something completely independent of cohesion. In addition to this the intensity of the frictional resistance is strongly influenced by dynamic lubrication phenomena. Thus the coefficient of friction between the point of the rapidly penetrating rod and the adjoining material is apt to amount to only a small fraction of the coefficient of friction between the same material and the surface of a slowly penetrating point. In turn, each of these two coefficients may be very different from the coefficient of friction between the point and a mixture of cement and water of equal consistency.

These friction phenomena are most troublesome wherever they develop in connection with experimental work. Due to the effect of dynamic lubrication it is impossible to establish a universally valid pile-driving formula. The same effect introduced an erratic element into the cone tests described in the first paragraphs of this paper. Since the outcome of Doctor Kindle's penetration test depends to a large extent on friction, the figures furnished by the tests can not possibly be considered as a standard for comparing consistencies. They represent the combined result of two independent factors—consistency and friction. The friction effect is complicated by dynamic lubrication and seems to have a far greater influence on the outcome of the tests than the consistency.

Three years ago the writer investigated the effect of the shape of the penetrating body on the ultimate bearing capacity of cohesionless sand[8] and found that the resistance against penetration of a cylindrical body with a flat bottom and with a diameter of 3.9 centimeters acting on the plane surface of the sand is about 20 per cent greater than the resistance against penetration of a cylinder of equal diameter, terminating in a cone with a vertex angle of 60°. On the other hand, several years ago, similar tests were made near Vienna, on cohesive loess.[9] The diameter of the loaded area was equal to 28.1 centimeters. In contrast to what has been stated above for the sand, the resistance of the loess against penetration of the conical bodies (vertex angle of 90° and of 35°) seemed to be somewhat in excess of the ultimate bearing capacity for a load acting on a plane surface. Hence the ratio between the bearing capacity of the ground for plane and for conical loaded surfaces may be different for different materials. This fact ought to be kept in mind when attempting the interpretation of cone penetration tests.

CUBE STRENGTH MOST PROMISING INDEX OF CONSISTENCY

From the preceding remarks we learn that the proposed methods for determining the "consistency" of a material are still far from ideal. For the time being the most promising method consists in measuring the cube strength directly, by a simple, nonconfined compression test, and to consider the yield point of the cubes as a measure of the consistency because, according to the aforementioned theory of plastic flow, the yield point of an unconfined cylinder strictly determines the ultimate resistance of a plastic material under any other conditions of confinement.

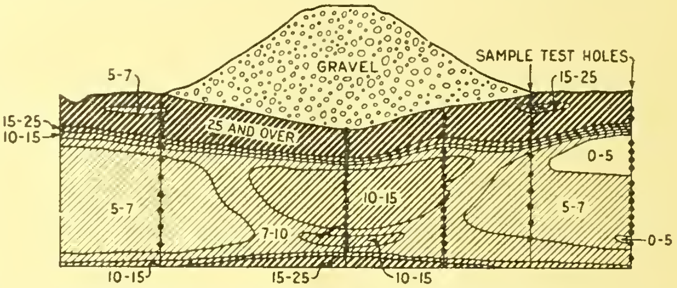

Fig. 7.—Diagram Showing Consistency Factors of Soil Beneath a Gravel Fill at Smedserod, Sweden. Copied from Report of Swedish Railroad Commission

It remains to decide to what extent a consistency coefficient thus obtained may serve a practical purpose. There are three principal fields of application for consistency tests:

(*a*) Applied geology, the consistency test serving as a means for expressing the hardness of materials (rocks, soils, muds, etc.) softer than talcum.

(*b*) Foundation engineering, the consistency test to be used for investigating the degree of stiffness of soil intended to support structures and artificial loads.

(*c*) Subgrade investigations, the consistency test to be used for expressing the resistance of the top layer of the ground, located immediately beneath the road surface.

The application of the consistency test to soft geological formations is severely handicapped by the great variation in the physical character of the softer formations. The common rocks and minerals included in the traditional scale of hardness represent practically homogeneous solid bodies. The homogeneous solid bodies belong, physically speaking, in one class, and their hardness can be described by a single figure.

In contrast to this, the soft materials may be either porous solid bodies (chalk, etc.) or two-phase systems (saturated silts, clays, etc.) or three-phase systems (dry clays or loams). These three groups of materials are physically so different that it is impossible to establish a scale of consistency valid for all of them. The hardness of the materials of the first and the third group may best be expressed by the cube strength (breaking strength) of the material or by any other method used for measuring the hardness of brittle solid bodies. The consistency of the materials of the second group is also primarily determined by the cube strength, but the cube strength has in this case a different meaning. It corresponds, not to the breaking strength of brittle substances, but to the yield point of plastic materials. Hence, in order to define the hardness of a soft material, it is not sufficient to mention the cube strength. It is necessary to state in addition, at least, whether the material was crushed or whether it flowed under pressure, and whether the material disintegrates in water or whether it retains its strength. For clays it will, in addition, be required to determine the consistency in an undisturbed state.

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[7] "A proposed scale of hardness and cohesion for rocks," an unpublished article by E. M. Kindle.

[8] Terzaghi, Erdbaumechanik, Wien, 1925, p. 232.

[9] From an unpublished report by Dr. Ing. Fritz Emperger.

The most promising field of application of the consistency tests concerns the stiffness and the bearing capacity of compressible substrata (muck, silt, clay) located beneath the depth of seasonal variations in water content. Figure 7 shows the change in consistency of a bed of silt produced by the superimposed weight of a gravel fill. The investigations which led to this diagram were made by the Swedish Railroad Commission, by means of the cone penetration method described in this paper. The diagram shows very clearly the general subsidence associated with an increase of the consistency of the strata located beneath the fill. Yet the data have a relative value only, because the tests have been made with remolded samples. At present our efforts concentrate on improving the methods for obtaining undisturbed samples and on testing these samples in their original condition.

When attempting to apply consistency methods to the problem of predicting the behavior of a subgrade to be located immediately beneath the surface of a road, one meets the same difficulties which make it impossible in the laboratory to go beyond an attempt to classify the raw material of the subgrade. In the field the subgrade undergoes periodical changes between the state of almost complete saturation and the state of dryness. As a consequence its consistency varies between very wide limits. In the preceding paragraphs it has been shown that the consistency of two identical samples with identical water contents may be very different according to the preceding history of the sample. It would be still more difficult to estimate in advance the minimum consistency of a material under conditions which we can not possibly reproduce in the laboratory. Hence in subgrade investigations the consistency tests can not possibly be utilized except in connection with laboratory procedure for classification purposes, and without expecting the test results to have a direct bearing on the behavior of the soil in the field.

(Continued from page 239)

hard bottom is reached, the bending stresses induced in the piling during backfilling can easily cause failure where sufficient penetration of the mineral subsoil can be had to give a good piling anchorage. If penetration of the piling is limited by a hardpan layer below the peat, sliding of the bottom of the piling may take place. If the deposit is deep, the piling may be flexible enough so that it acts as a cantilever fixed at the bottom and the whole structure may be moved in one direction. The forces due to the compression and flow of the peat during filling have been the cause of a number of bridge failures.

In one case in Michigan in 1916, a 40-foot bridge was put in, supported by 50-foot spliced piling. Some time after filling of the approach was started the bridge began to move, slid along the axis of the roadway and inside of a week had completely disappeared from sight. In another case in a peat deposit about 40 feet deep, a 40-foot bridge rotated about 5° about one end, and settlement then began in sufficient amount to destroy the bridge. Had the approach fills been made first, these failures would not have occurred.

The maintenance of a road surface which is settling is a comparatively difficult matter to handle. In a number of cases where the amount of settlement was not over a foot or two and a concrete surface had been placed, the slabs have been jacked back to grade and the space beneath the slab refilled with tamped earth. It is for this reason that mesh reinforcement is desirable, as it has been found that reinforced slabs can be jacked back into place with very much less breakage than the plain slab. In other cases, where the settlement has been accompanied by considerable slab breakage, it has been necessary to remove the surface, bring the fill back to grade, and replace the surface. Where the amount of settlement is considerable it may be possible to effect a remedy by removing only a part of the pavement at each end of the settlement and placing new sections which will join the sunken slab properly to the surface adjacent to the settlement.

It is unfortunate that the present methods of filling have been used for so short a time, as it is impossible for us to accurately estimate the amount of filling material necessary with methods 4 and 5. Quantities in each case will be less than those indicated by curves derived from an investigation of fills which have been placed full width on the surface of the undisturbed peat. It is planned in the near future, as soon as a sufficient number of these examples are available, to cross section and determine the amount necessary, using the new method. We are confident that the methods proposed will eliminate a great deal of the settlement formerly encountered, and will involve the use of smaller amounts of material in filling.

CALIFORNIA ROAD SURVEY DEMONSTRATES THE ECONOMIC POSSIBILITIES OF SUBGRADE STUDIES

Reported by C. A. HOGENTOGLER, Highway Engineer, Bureau of Public Roads

EXAMINATION of the data on subgrades and road condition collected in the study of California highways in 1920[1] shows convincingly the value of considering subgrade conditions in highway design.

In this survey the concrete roads were classified according to condition as follows:

(*a*) A pavement in which the plainly visible tranverse cracks do not exceed the normal number expected of a pavement constructed without expansion joints, and which has no plainly visible longitudinal cracks.

(*b*) A pavement having more than the normal number of plainly visible transverse cracks or with some "crowfoot" cracks at the edges or with both.

(*c*) A pavement similar to classes *a* and *b* and with one plainly visible longitudinal crack or with a considerable number of "crowfoot" cracks.

(*d*) A pavement so cracked transversely and longitudinally that numerous slabs are formed of less area than class *c*, but that do not average less than about 50 square feet.

(*e*) A pavement in which the plainly visible transverse and longitudinal cracks are so numerous that it is broken into slabs having areas less than about 50 square feet but in which no general disintegration appears.

(*f*) A pavement badly broken and with disintegrated portions.

The subgrade classification was made in accordance with the practice of the U. S. Bureau of Soils as follows:

Class 1. Clay and adobe soils (includes clay, silty clay, clay-loam and clay).

Class 2. Marsh lands (includes salt marsh and peat lands).

Class 3. Loams (includes loam, clay-loam, silty loam and silty clay-loam).

Class 4. Sandy loam (includes coarse sandy loam and fine sandy loam).

Class 5. Sand, and sand and gravel.

TABLE 1.—*Condition classification of concrete roads on different types of subgrade, based on study of 1,200 miles of California road in 1920*

Type of subgrade[1]	Class of pavement					
	A	B	C	D	E	F
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Sand	68.4	21.4	10.2	----	----	----
Sandy loam	44.2	38.8	11.0	3.6	0.9	1.5
Loam	34.8	39.0	18.1	4.2	1.1	2.8
Adobe and clay	21.4	37.8	20.4	10.8	4.6	5.0

[1] Types classified in accordance with the practice of the U. S. Bureau of Soils.

In the California report, about 1,200 miles of concrete surface and the corresponding subgrade were classified according to the above systems, with the result shown in Table 1. These roads were practically all of the standard California 4½-inch cross section used prior to 1920. They were constructed during the period 1912 to 1920 and the construction was distributed over different types of soils in approximately proportionate amounts throughout this period.

In the large mileage of pavement inspected and classified there were sections of various ages from one to eight years and sections which had been subjected to traffic of various degrees of density from light to heavy on each of the types of subgrade. In other words, there were pavements laid on sand subgrades which had been in service for varying periods, and there were pavements laid on adobe and other types of soil which had also been in service for various lengths of time; similarly, there were pavements on sand subgrades which had been subjected to light traffic and others which had been subjected to heavy traffic, and the same may be said of the pavements laid on adobe and other types of soil. In view of these conditions and the large mileage involved it may be assumed that major differences found to exist in the percentage and degree of failure of pavements on the various types of subgrades were due in greater degree to the character of the subgrade than to the age of the surfaces or the density and weight of the traffic.

There may be some differences of opinion as to the details of road and soil classification, whether the road condition classes should be based on the spacing of cracks and area of resulting blocks or on length of cracks and extent of breakage, and whether the soils should be classified according to type or according to certain laboratory tests. Whatever the opinion in regard to these matters it can influence but little the outstanding conclusions to be drawn from the comparative data in Table 1. Classification by other proposed systems which definitely identify road condition and soil characteristics would warrant the same conclusions.

These conclusions are as follows:

1. That subgrade character has a controlling influence on road condition and outweighs the traffic variable in general comparisons.
2. That a surface design which is adequate for one type of soil may be inadequate for an inferior type.
3. That within a given soil type certain undesirable conditions of subgrade may develop which will make inadequate a pavement design adequate where these conditions do not exist.
4. That, other things being equal, these undesirable subgrade conditions increase with increase of clay content.
5. That soils can be grouped with regard to the occurrence of these undesirable conditions of support, although the best classification may not yet be determined.
6. That information on the relative tendency of the various soil types to develop undesirable subgrade conditions and on the causes and corrective measures for

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[1] Report of a study of the California highway system, 1920.

these undesirable conditions can result in enormous savings in the ultimate cost of roads.

Table 1 indicates that, in general classifications, the traffic is subordinate to the subgrade variable. While no correction was made for differences in traffic, the percentage of good pavement consistently decreases and the percentage of bad pavement correspondingly increases as the supporting value or some other desirable characteristic of the subgrade decreases.

In comparing but two roads, difference in traffic undoubtedly will be reflected, but when large portions of highway systems are used for study the traffic variable seems subordinate.

This coincides with the findings of the highway research board which were to the effect that the influence of traffic on concrete pavement condition was subordinate to those of age, subgrade and design of slab.[2]

Table 1 demonstrates that a design of pavement which is adequate on sand subgrades may be inadequate on clays. This can be better demonstrated by a second tabulation. Since the California report, in further explanation of the road condition classes, states that classes A, B, and C were equally serviceable for traffic, and also since the adoption of the center joint design would place class C pavements with classes A and B, these three classes are grouped in Table 2 as desirable pavements and classes D, E, and F are grouped as undesirable pavements.

Table 2.—*Pavements on different types of subgrade classed as desirable or undesirable*

Type of subgrade	Desirable	Undesirable
	Per cent	*Per cent*
Sand	100.0	
Sandy loam	94.0	6.0
Loam	91.9	8.1
Adobe and clay	79.6	20.4

Table 2 shows clearly that the design of pavement used was adequate for sand subgrades but that other subgrades require either a change in design or corrective measures in varying amounts. This indication is supported by findings of the highway research board and also by behavior of certain roads in both New York and Delaware.

Table 2 indicates that road failures are caused by certain undesirable conditions found in portions of the various soil types. Even on the worst subgrade (clay) about 80 per cent of the pavements were in desirable condition. A further study of this table indicates that but three soil classifications are needed in this case and the revision is shown in Table 3.

Table 3.—*Pavements classed as desirable or undesirable using only three subgrade classifications*

Type of subgrade	Pavement condition	
	Desirable	Undesirable
	Per cent	*Per cent*
Sand	100.0	
Loam	93.0	7.0
Clay	79.6	20.4

This tabulation plainly indicates that the undesirable conditions increase with increase of clay content and also that soils can be classified with regard to their influence on pavement condition, thus supporting indications 4 and 5.

As an example of the value of subgrade information to the engineer let it be assumed that 1,000 miles of pavement are to be constructed; that the subgrades consist of 200 miles of sand, 300 miles of loam, and 500 miles of clay, and that conditions are similar to those existing in California.

In the absence of definite information on subgrade influence, a pavement of uniform design and adequate to meet the undesirable conditions in the clays and loams would probably be adopted. Referring to Table 3, this means that on the loams a pavement more than adequate for 93 per cent of its length would be used because of undesirable conditions existing on but 7 per cent, and in the clays a pavement more than adequate for 80 per cent of its length must be laid because of undesirable conditions found on 20 per cent, or under the assumptions stated above, 879 miles of our total length would be more than adequately improved because of undesirable conditions existing on 121 miles. Under the above conditions and with an assumed pavement cost of $21,000 per mile, the cost of our system would be $21,000,000.

In contrast to this procedure, if the highway engineer had definite information on the types of soils in which undesirable conditions exist and also on the location, cause and corrective measures for these conditions, a design adequate for the 879 miles of pavement would be used and corrective measures would be added on the 121 miles.

Just what these corrective measures would cost is not known, but certain roads in Delaware and New York indicate that the difference in support between natural sands or sands and gravels and undesirable conditions found in clay subgrades can be equivalent to at least 1 inch of pavement thickness or about $3,000 per mile. If this indication is correct, it can be seen that on the 879 miles of pavement there exists the possibility of saving $2,637,000 or 12½ per cent of the total cost. Whether or not these actual figures are accepted, the California data plainly show that if accurate subgrade information is obtained and utilized a considerable saving of road funds will result.

These data also point out that the logical method of procedure in subgrade studies is to make:

1. General surveys for determining the influence of various types of subgrades on different types and designs of pavements.
2. Detailed investigations of road failures to determine the causes of undesirable conditions developing in the several soil types.
3. Detailed surveys of subbases, subgrade treatments, drainage types, and change in pavement design to determine the efficiency of the various corrective measures for overcoming undesirable subgrade conditions.

At present 15 States consider subgrade support in pavement design, a fact which reflects increased recognition of the importance of the subject. Practice in Colorado can be described as a good example of the practical application of the present knowledge of

(Continued on page 251)

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[2] Proceedings of the Fifth Annual Meeting of the Highway Research Board, 1925, Part II.

DESIGN OF A CONSTANT TEMPERATURE MOIST CLOSET

By WALLACE F. PURRINGTON, Materials Engineer, New Hampshire Highway Department

THE TESTING laboratory of the New Hampshire State Highway Department recently found it necessary to design a moist closet to meet the conditions of the new specification of the American Society for Testing Materials, which reads as follows:

"The temperature of the room, the materials, the mixing water, the moist closet and storage tank water shall be maintained as nearly as practicable at 21° C. (70° F.) and the mixing water, moist closet, and the water in the storage tank shall not vary from this temperature more than 3° C. (5° F.)."

This is a much more rigid requirement than that in the older specification, which stated that the air and water should be maintained as nearly as practicable at a temperature of 21° C. (70° F.) and did not give a definite tolerance limit. Testing laboratories may find it necessary to install new equipment to meet the new conditions. In the Northern States the problem is to maintain the necessary amount of heat in the fall and spring, while in the Southern States the greatest difficulty is in reducing heat in summer.

In designing a moist closet for New Hampshire conditions the following features were considered—

1. Insulation against heat and cold.
2. Equipment to generate and control the necessary heat.
3. Equipment to maintain the maximum degree of humidity.

The closet sidewalls were constructed of 6-inch hollow tile and lined with 1½-inch cork board coated with a bituminous mastic one-quarter inch thick. The cork board was cut into slabs 12 by 36 inches in size and cemented to the tile by a rich cement-sand mortar. The cork was treated with a chemical sealer to prevent staining by the mastic. Three coats of a cement-stucco paint were applied to the mastic coat after the wall-board was placed. The room has been in operation for three months and there is not the least sign of

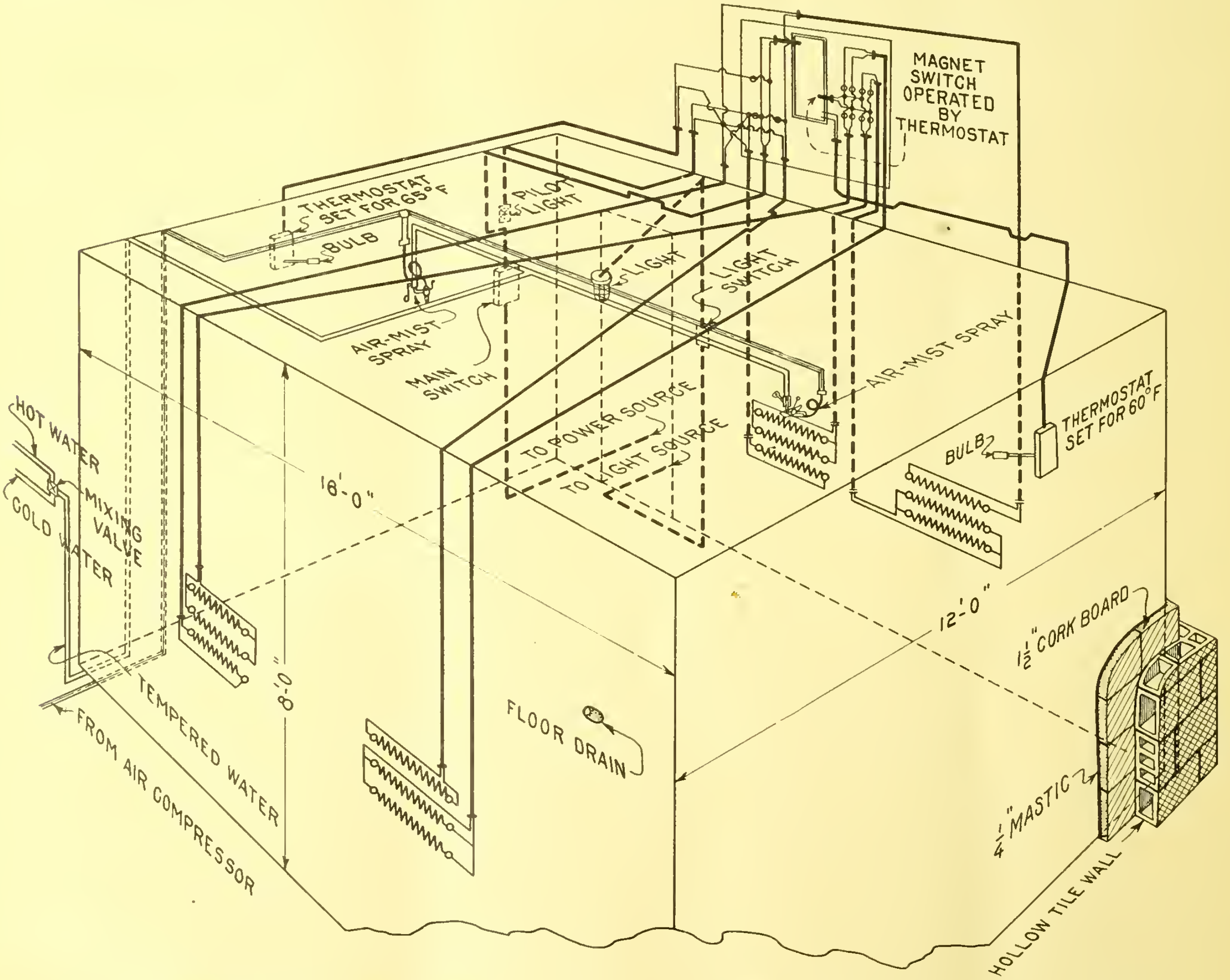

Fig. 1.—Sketch Showing Arrangement of Temperature and Moisture Controls for Moist Closet

the mastic causing discoloration. The floor is made of concrete, and the top of the closet is constructed of 4 inches of reinforced concrete. The door is of wood and is lined with cork. This construction has been found to give entirely satisfactory insulation.

HEAT CONTROL EQUIPMENT

Two possible sources of heat were considered, steam from the heating plant of the building and electricity. It was decided that steam heat could not be regulated satisfactorily, while electric heating could be easily controlled, and in addition would be available at all times, although more costly.

The heating units used were 500-watt capacity heaters. Four of these were connected in series and eight in parallel. The series group was connected to a liquid-type thermostat set at 65° F. and the parallel group was connected to a similar thermostat set at 60° F. The latter were wired through a 110-volt, 60-ampere remote control switch. But little use has been made of this group as the temperature has not fallen to the point where it comes on. Figure 1 shows that the heating units are arranged in four groups, each consisting of one of the units in series and two of those in parallel. It is necessary that the heating units, switches, thermostats, and connections be of the best type and not affected by corrosive action in the warm humid atmosphere. These heaters require considerable current and it is necessary that the wires and insulation be of the very best. The main feed consists of No. 6 rubber-covered wire while all outside wiring is No. 4 common covered wire. Owing to the excessive heat and dampness, the wires connecting the fuse block and control switch are No. 14 and of the slow-burning type.

At the present time a recording thermometer has not been installed and it is necessary to take readings at different periods of the day. Typical observations covering a week have been recorded as follows:

Date	Temperature in degrees Fahrenheit		
	8.30 a. m.	12 m.	5 p. m.
Nov. 15	68. 7	66. 9	66. 9
Nov. 16	66. 2	68. 5	70. 7
Nov. 17	70. 1	68. 9	67. 5
Nov. 18	65. 7	68. 0	68. 0
Nov. 19	67. 1	69. 1	67. 1
Nov. 20	66. 9	67. 1	----------

EQUIPMENT TO MAINTAIN HUMIDITY

Two humidifiers supply a very fine fog spray. They are connected with water and compressed air. By adjustment it is possible to get nearly any degree of humidity required. During the fall and winter the water is very cold. Unless it is tempered somewhat by the use of warm water and a mixing valve there is a great demand on the heaters to keep the room to the desired temperature. The purpose of the mixing valve is simply to reduce the expense of operating. The heaters will keep the temperature up, however, if tempered water is not introduced. The humidifying apparatus also serves to keep the atmosphere in the chamber in constant motion and at a uniform temperature throughout.

Pans for the storage of briquettes, and 2 by 4-inch cylinders have been installed in this room. The pans are arranged in such a manner that a small trickle of water empties into a holding tank which allows the water to come to the temperature of the room and the water overflows into a pan in which briquettes are stored. In the second pan there is an overflow pipe which empties into another pan below. Five pans, including the storage pan, are placed in this way and are supported by a frame made of angle irons, the joints of which have been electrically welded. This was done to avoid the rusting of screws, bolts, and washers. Two pans much longer and deeper than the one described above are used for the storage of cylinders. The room is large enough to store all the concrete cylinders the laboratory will be called upon to test.

A large stock bottle of distilled water for mixing is kept in this room at all times. Enough is drawn off for each test and as a result the requirement of keeping the mixing water within the tolerated limits can be carried out. Immediately upon making the pats for initial and final set and for the steam test they are placed in this room upon a slate-covered bench and are kept there during the period required.

(Continued from page 249)

subgrades. Prior to construction a detailed survey is made of all subgrades and soil samples are obtained for laboratory test. These test results then determine both the locations for and character of treatment. Subgrade treatments of sand are provided for all locations in which the lineal shrinkage exceeds 5 per cent and the thickness of the sand layer used depends on the extent by which the moisture equivalent exceeds 20 per cent. This is determined by reference to a curve which shows the percentage of sand required to reduce the moisture equivalent from any amount to 20 per cent. While the method of treatment and the limits of 5 and 20 per cent might be subject to variation, the practice of using subgrade information forecasts its utilization in the future on all highways.

EFFECT OF HIGHWAY SLASH ON INFESTATION BY WESTERN PINE BEETLE STUDIED

The Bureau of Entomology of the United States Department of Agriculture has recently completed a four-year investigation of the effect of slash from highway clearing on insect infestation of surrounding timber. The work was conducted by J. E. Patterson, assistant entomologist, on a highway in the Cascade Mountains of southern Oregon.

It has been known that slash in all sections of the country is usually infested by one or more species of two groups of forest insects commonly known as bark beetles and borers. Until recently, however, little had been done to make a scientific analysis of the relation between such infestations and infestations in surrounding timber.

The Bureau of Entomology began its study in 1920 with the clearing of a highway through a larger timber area consisting of yellow pine, sugar pine, Douglas fir, white fir, and true fir. The slash consisted of entire trunks and tops of trees dragged to either side of a clearing 60 feet wide and 24 miles long. In this region the western pine beetle is the principal insect enemy of the mature western yellow pine.

To investigate the relationship of the slash to the attack and breeding of bark bettles, 1 square foot of bark was taken from the middle of the butt cant of each felled tree after the emergence of the broods and entrance and exit holes counted. These data were then compared with data from standing trees which had been attacked by bark beetles.

The conclusions were as follows:

Line slash of the character here considered is very attractive to the bark beetle *Dendroctonus brevicomis*, practically all such slash being attacked by this insect.

The attack of this bark beetle on the slash is not so heavy as its attack on mature standing timber. In the particular case studied approximately one-half as many beetles attacked a unit area of bark on the felled trees as attacked an equal area on standing timber.

The broods developing in slash are characterized by abnormal mortality. The increase of beetles developing in the slash studied was only 64 per cent of the number of beetles making the attack, whereas the corresponding increase in adjacent standing timber at the same time was 135 per cent.

Bark beetles from the surrounding standing timber are attracted to the slash at the time of attack, and a temporary concentration of infestation occurs in its immediate vicinity. Normal distribution of the infestation is resumed within a year.

The concentration just mentioned and the breeding of beetles in line slash do not increase or greatly influence infestations in the surrounding forests. The cycle of an infestation continues regardless of the slash.

This study indicates that the infestation of line slash by *Dendroctonus brevicomis* is not a serious menace to neighboring mature timber, and may be disregarded when the problem of slash disposal is under consideration.

THE ACTION OF CALCIUM CHLORIDE ON CEMENTS[1]

Some interesting observations as to the action of calcium chloride on cements are contained in an article by M. Anstett in Le Génie Civil for October 2, 1926. This article is the result of an investigation on the corrosive action of calcium chloride on steel reinforcement.

It was found that very dilute mixtures of calcium chloride, such as a few grams per liter of mixing water, retarded the setting of the cement quite noticeably. Concentrated solutions, such as 100 to 400 grams per liter, accelerated the setting and hardening. The calcium chloride admixture is a stabilizer of mortar, due no doubt to its affinity for free lime such as is present in varying amounts in all cements. The elimination by combination of this free lime without delay removes a frequent cause of swelling and disintegration of concrete.

Mixed with a strong solution of calcium chloride, the cement acquired considerable compressive strength in a very short time, but samples of a similar mixture disintegrated as a result of immersion in water a few moments after setting. Other specimens that had cured in air for 15 to 20 hours remained intact during submergence in water. Other factors are known to influence the effect of calcium chloride.

To illustrate the influence of this admixture on the soundness, 18 cement pats were prepared, 9 of which were mixed with pure water, and the remainder with 5 per cent by weight of calcium chloride. After curing for a few days in water at 15° C., 3 of the 9 pats mixed with pure water had developed cracks, but all of those with the admixture were intact.

Efflorescence increases when calcium chloride is added in strong proportions, due, no doubt, to the deliquescence of the chemical and the constant humidity of the cement as thus maintained.

Table 1 illustrates the influence of calcium chloride admixture on the time of setting.

TABLE 1.—*The influence of calcium chloride admixtures on the time of setting of cement*

Cement	Cement mixed with pure water		Cement with about 3 per cent of $CaCl_2$		Cement with about 6 per cent of $CaCl_2$	
	Initial set	Final set	Initial set	Final set	Initial set	Final set
	H. m.	*H. m.*	*H. m.*	*H. m.*	*H. m.*	*H. m.*
Aluminate	4 10	6 40	0 20	1 05	0 12	0 35
Holderbank	3 ..	6 45	0 16	1 20	0 04	0 11
Ordinary Portland	3 15	6 45	0 15	2 30	0 04	0 17
Slag	3 15	19 ..	3 45	11 30	1 45	6 15

It must be observed in passing that the 6 per cent admixture is not to be recommended for use with the Holderbank and ordinary Portland cement, as the four-minute time of initial setting is too short a period to employ in practice.

Table 2 indicates the resistance to compression in pounds per square inch, and the influence of calcium chloride as an admixture.

TABLE 2.—*The influence of calcium chloride admixtures on the compressive strength of cement*

Cement	Test specimens cured					
	In water at 15° C.				In air at minus 20° C.	
	Mortar mixed with pure water		Mortar mixed with 6 per cent $CaCl_2$		Mortar mixed with 6 per cent $CaCl_2$	
	2 days	7 days	2 days	7 days	2 days	7 days
	Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
Aluminate	3,300	4,600	2,350	2,370	1,720	2,480
Holderbank	950	2,740	2,340	2,510	1,340	3,020
Ordinary Portland	336	1,510	1,070	1,770	700	1,316
Slag	266	826	28	168	28	826

The admixture of 6 per cent calcium chloride was apparently harmful where the temperature of the bath was 15° C., except for ordinary Portland cement. It is of some advantage where exposed to temperatures as low as minus 20° C., for the specimens mixed with pure water developed practically no strength at this temperature.

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[1] Translated and abstracted by C. S. Jarvis, associate highway engineer, Bureau of Public Roads.

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Report of the Chief of the Bureau of Public Roads, 1925.
*Report of the Chief of the Bureau of Public Roads, 1926. 5c.

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Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JANUARY 31, 1927

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,726,985.09	1,298.3	$ 1,834,960.34	$ 889,114.86	101.9	$ 5,519,632.52	$ 2,604,130.30	296.7	$ 141,817.16	$ 70,308.93	13.3	$ 3,606,535.83	Alabama
Arizona	10,949,878.25	5,863,772.36	729.8	621,097.34	352,011.23	35.8	1,343,390.41	919,184.58	75.9	22,957.56	16,607.49		3,522,667.36	Arizona
Arkansas	18,364,544.60	7,666,698.35	1,323.0	1,761,995.60	806,926.30	115.6	3,735,948.91	1,816,344.33	248.6	1,448,975.67	715,201.88	95.2	1,880,529.14	Arkansas
California	27,142,696.90	13,003,592.30	1,058.0	6,242,897.06	2,542,537.22	171.4	9,088,262.84	4,404,837.95	194.2	412,370.87	167,114.97	8.4	4,438,169.56	California
Colorado	13,905,904.64	7,127,288.18	745.0	958,040.35	522,177.14	37.9	5,912,091.76	2,878,868.76	266.8	856,076.31	143,620.34	20.1	3,030,377.58	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1	684,812.28	245,719.74	13.7	5,699,574.28	1,580,996.95	70.4				879,063.51	Connecticut
Delaware	4,918,052.29	1,781,666.60	124.3	1,038,483.07	452,067.18	28.0	591,038.06	241,755.90	17.6	183,700.71	87,799.94	7.1	276,394.38	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	2,120,466.67	1,049,833.66	62.7	9,007,138.20	4,040,758.40	229.1	1,135,928.26	415,082.24	24.7	1,654,368.39	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	4,266,434.44	2,102,653.99	255.7	9,346,042.77	4,573,997.18	406.0	199,260.48	99,680.23	4.4	1,970,592.77	Georgia
Idaho	11,061,198.14	5,882,112.70	724.7	1,920,786.49	1,055,775.24	102.9	2,054,615.81	1,291,376.25	162.6	208,283.78	130,082.19	13.0	1,135,473.62	Idaho
Illinois	44,116,611.86	20,619,995.74	1,377.7	2,406,008.82	1,177,396.86	81.9	9,740,785.37	4,668,106.54	347.8	665,850.50	326,680.26	24.1	6,195,447.60	Illinois
Indiana	16,949,425.87	8,172,126.19	534.3	3,638,469.22	1,738,093.43	116.9	16,204,004.17	7,590,742.41	456.4				2,630,165.97	Indiana
Iowa	29,062,376.40	11,926,302.10	2,114.8	3,731,392.31	1,720,815.87	233.3	11,897,224.91	5,766,996.30	616.0	3,493,361.51	1,683,798.77	94.5	433,649.96	Iowa
Kansas	32,826,601.64	12,590,489.26	1,160.6	1,762,046.93	792,649.00	142.7	13,306,357.44	6,284,820.81	721.3	2,516,823.08	883,972.21	93.4	1,981,011.73	Kansas
Kentucky	20,737,706.10	8,492,082.26	758.3	1,759,367.93	692,505.85	78.0	7,574,192.37	3,602,891.90	357.3	472,126.06	229,167.79	25.6	1,614,108.21	Kentucky
Louisiana	13,830,692.68	6,144,739.99	1,056.9	886,239.31	393,861.28	31.8	4,824,231.36	2,307,333.02	220.6	435,627.20	197,677.90	11.2	1,242,103.87	Louisiana
Maine	8,747,652.76	4,192,507.39	303.6	1,461,181.62	538,004.58	43.5	2,612,258.85	951,262.17	73.9	87,912.01	43,956.00	5.6	1,419,891.86	Maine
Maryland	10,924,943.10	6,112,991.22	423.3	356,216.42	168,190.63	24.2	1,376,382.60	640,064.42	64.2				638,929.73	Maryland
Massachusetts	18,363,757.71	6,657,260.62	374.5	412,120.06	121,949.75	5.1	5,416,681.36	1,483,354.92	78.1	1,085,305.70	308,090.10	20.7	2,627,170.61	Massachusetts
Michigan	26,997,240.78	11,827,062.30	963.0	1,324,273.80	612,755.76	43.1	14,266,486.78	6,461,672.97	403.9	533,994.00	218,610.00	14.6	3,437,064.98	Michigan
Minnesota	37,170,986.96	15,586,116.56	3,181.9	7,860,229.10	3,430,029.11	461.6	3,514,912.74	775,498.90	212.2	3,214,512.67	357,000.00	159.3	1,563,876.43	Minnesota
Mississippi	16,146,088.62	7,414,534.10	1,129.0	1,168,069.25	581,407.96	74.0	7,985,181.01	3,895,056.87	406.5	196,624.31	98,312.15	17.3	1,446,585.92	Mississippi
Missouri	28,989,166.92	13,736,014.86	1,543.2	9,587,678.62	4,204,228.49	284.4	11,598,617.86	4,607,458.42	313.0	2,007,316.24	899,956.78	105.3	1,743,892.46	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	1,294,991.62	857,209.60	82.8	1,575,476.67	1,353,362.30	130.3	856,661.87	452,047.21	90.2	5,980,299.00	Montana
Nebraska	11,533,401.62	5,474,202.52	1,768.3	2,334,199.91	1,152,552.23	258.3	12,690,828.64	6,182,149.79	1,331.4	430,139.87	203,245.97	40.2	3,208,222.49	Nebraska
Nevada	7,558,195.51	5,130,934.59	638.8	2,418,585.71	2,078,456.00	246.9	1,612,460.46	1,393,756.26	210.9				1,140,578.15	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6	520,363.47	241,809.41	17.8	1,000,087.73	453,480.17	27.3				462,377.35	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	5,881,939.07	2,397,022.27	26.0	4,600,662.42	855,223.56	55.0	331,087.93	104,910.00	7.0	946,532.96	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0	36,773.39	21,877.50	0.6	3,248,332.74	2,239,874.76	278.6	282,955.23	236,525.13	9.3	2,321,214.23	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	4,686,404.51	1,678,684.42	107.6	38,720,873.00	10,025,600.20	628.6	4,697,900.00	1,060,797.50	57.6	7,003,372.69	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9	5,677,359.00	2,274,655.41	124.7	4,380,616.71	2,025,476.48	135.6	490,853.80	238,440.59	20.1	1,714,652.58	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	3,323,775.06	1,610,686.24	486.2	4,611,036.04	2,517,465.95	676.3	672,701.96	336,351.42	80.1	1,447,246.61	North Dakota
Ohio	47,699,632.90	17,371,787.03	1,364.1	3,298,360.96	1,316,121.47	112.8	12,291,043.88	4,720,402.19	362.8	1,377,279.96	542,093.66	17.9	4,543,600.65	Ohio
Oklahoma	28,247,950.33	13,159,999.16	1,178.9	1,177,814.08	653,488.56	47.0	3,686,908.89	1,511,084.32	201.1	1,624,340.02	558,021.83	90.3	2,029,084.14	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	702,784.65	403,640.92	23.7	3,278,772.59	1,816,407.15	127.8	409,163.92	215,220.95	12.0	1,033,065.19	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1,188.8	838,497.58	216,460.04	14.5	29,574,896.82	8,442,326.00	591.6	2,722,312.77	1,008,481.91	59.3	3,446,516.01	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	274,781.19	113,520.00	7.6	2,024,903.87	531,795.00	35.5	839,058.07	74,175.00	4.9	754,874.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1,481.9	1,694,832.67	646,425.42	75.3	5,304,063.44	2,299,639.32	186.7	99,046.24	42,921.58	10.3	1,101,202.75	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,181.2	1,331,176.93	664,464.75	222.7	3,422,826.30	1,734,580.84	610.2	220,867.18	89,849.59	23.8	1,294,131.85	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	1,589,779.61	747,492.21	49.1	9,062,838.88	3,936,706.92	258.8	424,027.84	80,000.00	7.7	1,854,573.85	Tennessee
Texas	69,183,673.46	27,440,264.72	4,920.2	5,240,853.93	2,366,551.01	299.9	17,330,477.66	7,811,296.00	741.2	3,317,531.66	1,505,050.36	79.2	5,980,550.91	Texas
Utah	8,253,178.03	5,098,440.68	546.4	825,999.14	615,614.06	79.1	1,666,310.69	1,262,142.56	129.9	734,811.96	563,860.83	42.7	1,125,626.88	Utah
Vermont	4,242,042.64	2,017,699.51	134.5	314,565.23	134,066.49	6.6	1,967,767.73	759,594.56	36.5				722,771.44	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.5	2,005,236.91	891,387.53	62.5	5,851,503.28	2,452,570.58	158.8	723,986.58	97,955.23	18.6	1,116,586.55	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	300,577.74	116,642.49	8.7	4,382,234.09	2,045,600.00	94.9	140,581.59	62,000.00	7.0	1,280,156.05	Washington
West Virginia	9,473,716.44	4,141,062.65	392.9	961,130.88	432,685.36	26.5	5,634,425.23	2,201,847.66	164.3	1,434,758.36	529,950.92	64.8	840,600.41	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1	894,671.96	416,931.79	45.0	8,191,712.72	3,915,680.88	363.0	64,419.42	32,209.71	2.8	4,551,741.89	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.5	1,370,684.55	875,427.00	150.3	1,977,666.96	1,251,602.63	135.8				1,332,726.32	Wyoming
Hawaii				343,664.15	97,440.00	6.6	1,439,661.16	464,352.64	23.1	346,662.73	98,010.00	6.5	805,976.36	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	106,032,067.23	49,111,077.21	5,134.8	352,033,430.94	146,589,426.94	13,936.8	40,359,973.64	15,214,439.56	1,510.7	107,406,352.71	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 112,824,428.82 Federal aid $ 46,788,763.64 Miles 4,098.0

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